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SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1

1a. REPORT SECURITY CLASSIFICATION UNCLASSIFIED	1b. RESTRICTIVE MARKINGS NONE
2	3. DISTRIBUTION/AVAILABILITY OF REPORT APPROVED FOR PUBLIC RELEASE; DISTRIBUTION UNLIMITED.
2	
4. AD-A217 865	5. MONITORING ORGANIZATION REPORT NUMBER(S) AFIT/CI/CIA-89-010
6a. NAME OF PERFORMING ORGANIZATION AFIT STUDENT AT OHIO STATE UNIVERSITY — 6b. OFFICE SYMBOL (If applicable)	7a. NAME OF MONITORING ORGANIZATION AFIT/CIA
6c. ADDRESS (City, State, and ZIP Code)	7b. ADDRESS (City, State, and ZIP Code) Wright-Patterson AFB OH 45433-6583
8a. NAME OF FUNDING/SPONSORING ORGANIZATION — 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code)	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. / PROJECT NO. / TASK NO. / WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification) (UNCLASSIFIED)
A Study of Four-Way Stop Intersections: The Validation of a Delay Model

12. PERSONAL AUTHOR(S)
NANCY BETH LYON

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
THESIS/[redacted]	FROM ______ TO ______	1988	135

16. SUPPLEMENTARY NOTATION
APPROVED FOR PUBLIC RELEASE IAW AFR 190-1
ERNEST A. HAYGOOD, 1st Lt, USAF
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17. COSATI CODES		
FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

DTIC
ELECTE
FEB 12 1990
S D
D

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT
☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS

21. ABSTRACT SECURITY CLASSIFICATION
UNCLASSIFIED

22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE (Include Area Code)	22c. OFFICE SYMBOL
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DD Form 1473, JUN 86 Previous editions are obsolete. SECURITY CLASSIFICATION OF THIS PAGE

THESIS ABSTRACT

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QUARTER/YEAR: Autumn/1988

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TITLE OF THESIS: A STUDY OF FOUR-WAY STOP INTERSECTIONS: THE VALIDATION OF A DELAY MODEL

The four-way stop intersection is a controversial method of traffic control but it has seen limited research and existing guidance is not specific. Richardson developed a delay model to predict average time in the system at four-way stop intersections but he does not validate it with field data. This shortcoming is investigated here by taking field measurements to evaluate the accuracy of the model predicitons. The results support the accuracy of the model. Using a current value for minimum allowable headway improves the accuracy of the model predictions.

This study also looks at how and why four-way stops are used in the field through interviews with practicing engineers. (SDW)

This thesis is 135 pages.
A bibliography is attached.

-A-

Advisor's Signature

A STUDY OF FOUR-WAY STOP INTERSECTIONS:

THE VALIDATION OF A DELAY MODEL

A Thesis

Presented in Partial Fulfillment of the Requirements for

the degree Master of Science in the

Graduate School of the Ohio State University

by

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* * * * *

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I would not have made it without you

ACKNOWLEDGEMENTS

I would like to take this opportunity to thank my advisor, Dr. Nemeth, for his support and guidance. Dr. Nemeth, I appreciate the time you spent with me. Your suggestions were beneficial; you have helped me develop a greater understanding of traffic engineering. I would also like to thank Dr. Merry and Dr. Rogness for their suggestions and advice.

I have to extend a special thanks to my fellow graduate students who helped me with my field work. Xiaoduan Sun, Paul Ma, Cori Kurtz, Wendy Hutchinson, and Deanna Fry, I truly would not have been able to do it without you. And Al Stadler, you helped me with more than just my field work and anlysis, you kept me sane. Thank you so very much for everything you did for me these past few months.

The Air Force provided me with the wonderful chance to continue my education. I appreciate the guidance I received from Major Ken Schnell and Lt Col Jim Dando, who helped me take advantage of this unique opportunity.

Finally, I thank my family. Your constant support means so much to me; I would not be where I am today if I did not grow up with your love and guidance. Thank you Dad, Miva, Scott and Debbie, I love you.

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INTRODUCTION

Urban intersections are a primary concern for traffic engineers; they are the location of many accidents and the cause of most urban delay. There are several methods of controlling traffic at an intersection. One method is the four-way stop. The four-way stop is highly disputed among traffic engineers as a method of traffic control. Despite the controversy, only a limited amount of research has been accomplished in this area.

Current guidance on the use of four-way stops is covered in Chapter 10 of the 1985 Highway Capacity Manual. This chapter contains a general discussion of multi-way stop intersection capacities. The capacity values are taken from limited research conducted in the early 1960s. The Manual gives no specific procedural guidance for determination of capacity at multi-way stops. This deficiency is identified as a significant shortcoming by the Transportation Research Board (TRB), since the Manual does include specific procedures on how to determine capacities at two-way stops and at traffic signals.

The TRB published Circular 319 in 1987, identifying desired areas of research within the field of transportation

engineering. It identifies a need for research of capacity and levels of service at multi-way stop controlled intersections. At the time of publication, no recent work had been conducted in this area and reported in the literature. The TRB says that since multi-way stops are prevalent in many parts of the country, providing a valid technique for determining delay, capacity, and levels of service at multi-way stops is a high priority goal. (TRB, 1987)

This study is conducted in an attempt to help achieve that goal. It is divided into three major areas of research. The first section covers the literature review. The second section involves validation of a four-way stop delay model developed in 1987. The third section deals with the use of four-way stops in the field. This section includes interviews with practicing traffic engineers (and other individuals involved in the decision-making process for sign installation) and a brief look at some legal cases involving the misuse of four-way stops. The combination of these various aspects of four-way stops gives a comprehensive overview of this type of intersection control device.

History

Research into the history of four-way stops uncovers some interesting patterns of opinion and use, which are

reflected in the studies conducted. The research usually focuses on vehicle delay at the four-way stops, but the influence of current public concerns is also evident in the chronology.

Literature in the late 1940s indicates a growing use of four-way stops as a method of traffic control. Traffic engineers are concerned about using the control device properly. A need for guidance regarding stop sign installation is identified but unfortunately no action is taken until a much later date.

Studies in the early 1950s concentrate on comparison of delays at four-way stops as compared to other methods of control, such as two-way stops and traffic signals. There is greater delay at semi-actuated traffic signals (Hall, 1953) and less delay at two-way stops (Keneipp, 1951) as compared to four-way stops.

Controversy over the use of four-way stops becomes evident in the late 1950s. Some engineers favor the use of four-way stops as a device to satisfy pressure groups demanding action at intersections that warrant no action. (Keneipp, 1951) Other engineers feel that using four-way stops to quell political pressure will result in serious overuse, causing unnecessary delay and fostering driver disregard for all stop signs (Hanson, 1957); studies are conducted that support this claim. (Wilkie, 1954)

Some communities were already using too many four-way stops, so they initiated stop sign removal programs. Peoria, Illinois was one of the first, and this city conducted a before-and-after study to evaluate the results of their stop sign removal program. (Hanson, 1957) From this study, Hanson compiled a list of advantages and disadvantages associated with the use of four-way stops and he proposed warrants to regulate their use. (Hanson, 1957) The warrants, which were based on traffic volumes and accident data, were subsequently adopted by the Institute of Traffic Engineers in 1959.

In 1963, Jaques Herbert conducted a study that later became the Highway Capacity Manual's sole basis of determination of four-way stop intersection capacity. The study involves the determination of capacities of four-way stop intersections under various traffic and operating conditions based on vehicle departure headways. (Herbert, 1963) The capacity values he establishes are included in the 1965 Highway Capacity Manual and again in the 1985 Manual.

No further attention is paid to four-way stops until the 1970s with the advent of the energy crisis. Four-way stops are re-evaluated with special attention given to fuel consumption, emissions, and safety as well as delay. (Hall et al, 1978)

In 1979 the computer age enters the area of traffic engineering with the development of the traffic experimental and analytical (TEXAS) simulation model. (Lee and Savur, 1979) This model allows traffic engineers to determine either level of service, intersection geometry, type of control, or volume of traffic accommodated when the other three factors are known.

Research in the 1980s continues to look at delays at four-way stops. (Briglia, 1982) Once again traffic engineers question the basic need for four-way stop control at intersections. (Carter and Chadda, 1983) By this time, jurisdiction officials have a better understanding of the costs associated with four-way stops. Many stop sign removal programs are initiated, but they differ greatly in method and procedure. Carter and Chadda develop a standardized stop sign removal process based on the successes and failures of the previously completed programs.

Despite this attention, traffic engineers have not researched capacity at four-way stops since Herbert's study in 1963. In 1987, Richardson re-directs attention to this area. He develops an analytical model to predict delay at four-way stops. (Richardson, 1987) His M/G/1 queuing model (negative exponential arrival rates, general distribution of service rates, and a single server) is able to reproduce results from the TEXAS simulation model but it does not require the same detailed inputs. The queuing model is

examined in more detail in the second section of this study.

Validation of the Richardson M/G/1 Queuing Model

Richardson developed an analytical model based on queuing theory to predict delay at four-way stop intersections. The only information required by the model is intersection volume data and the number of lanes in each approach. Service time, or departure headway, is predicted through the use of headway values obtained from the 1963 Herbert Study. These values are incorporated into the model to predict average time in the system for each approach of the intersection. Richardson's M/G/1 queuing model predictions show strong agreement with predictions made by the TEXAS simulation model. However, he does not support the model with field data.

This study takes field measurements for the variables used by the M/G/1 queuing model to test the reliability of its predictions of delay. It also checks the accuracy of the average departure headway values taken from the Herbert study. Statistical comparisons are made to determine how well the queuing model predicts actual field events.

Field measurements were taken at three intersections in the Columbus, Ohio metropolitan area. The measurements are used to check the average service times of left turning, right turning, and through vehicles for the three types of intersection loads described in the Herbert study. The

first intersection load type is L, loaded, where at least one cross-street vehicle enters the intersection before the vehicle at the stop line of the approach being studied enters the intersection. The second type is N, non-loaded, where there are no vehicles within 50 feet of the intersection on either cross-street approach. The third type is I, interference, where a vehicle is present within 50 feet of the intersection on either cross-street approach but it does not proceed into the intersection before the vehicle on the approach being studied enters the intersection.

The results show that service times measured in this study are significantly different than the service times reported in the Herbert study. Today's service times are almost half the 1963 values in all reported categories. The value critical to the Richardson queuing model is the measured minimum allowable headway, or the average service time at the intersections with load type N. Herbert measured this value as 4.0 seconds. This study measured minimum allowable headway as 2.58 seconds. The effect of this difference on the model predictions is examined later in the study.

The validation of Richardson's M/G/1 queuing model consists of two basic parts because of the model's prediction method. The first part uses the Pollaczek-Khintchine formula to predict the average time in the system

on an approach of a four-way stop. This formula says:

$$L = [2P - P^2 + \lambda^2 V(S)] / [2(1 - P)] \quad (1)$$

where

L = average number in the system (average number on the approach, including the vehicle at the stopline);
λ = average arrival rate;
S = average service time (= Herbert's departure headway);
V(S) = variance of service time; and
P = utilization ratio (arrival rate*service time).

Using Little's equation, the average time in the system is then determined by

$$Ws = L / \lambda \quad (2)$$

where Ws is the average time in the system.

Field measurements were taken for each variable in the Pollaczek-Khintchine formula. The measured values were substituted into the equation to obtain the predicted average delay in the system. These predictions are compared to the measured field delays to test the accuracy of the formula. The normal and "t" statistical tests show that there is no significant difference between the formula predictions and the actual delay that occurs in the field.

The M/G/1 queuing model only requires traffic volume and the number of lanes in each approach of the intersection as input data. Obviously, this is not enough information to satisfy the Pollaczek-Khintchine formula. The average arrival rate is approximated by the volume count

information, but the average service time and its variance are predicted by the second part of the queuing model. The necessary equations are based on probability theory and the assumption of equal service time on opposite approaches.

The equations used at a simple four-way stop with one lane per approach are:

$$Pns = 1 - (1 - Pn)*(1 - Ps) \tag{3}$$

$$Sn = t(m)*[1 - Pew] + T(c)*[Pew] \tag{4}$$

$$V(S) = t(m)^2 * [T(c) - S] / [T(c) - t(m)] + T(c)^2 * [S - t(m)] / [T(c)-t(m)] - S^2 \tag{5}$$

where

- Pns = utilization ratio on the north-south approaches [Pew is similarly defined for east-west approaches]
- Pn = utilization ratio on the north approach (service time on the north approach times the approach volume divided by the total volume) [Ps is similarly defined]
- Sn = service time on northbound approach [Ss, Se, and Sw are similarly defined]
- t(m) = minimum allowable headway, 4.0 seconds
- T(c) = total intersection clearance time, 15.2 seconds
- V(S) = variance of the service time

Similar equations are developed for Pew, Ss, Se, and Sw. These equations are used to predict service time and its variance for the approaches of the three intersections studied. The predictions are used in the Pollaczek-Khintchine formula to obtain predictions of average time in the system on each approach. These predictions are compared to the measured values to evaluate how accurately the M/G/1 queuing model predicts field delays. Statistical analysis

using the paired "t"-test shows that the model predictions are not significantly different from field values for average time in the system, at a 95 percent level of confidence.

The M/G/1 queuing model predictions are also evaluated using the revised value for minimum allowable headway in the average service time and variance of service time calculations. Herbert reported a value of 4.0 seconds as the average headway when there is no traffic on the cross-street (load type N). However, this study obtains a minimum average headway of 2.58 seconds. This value is substituted into the equations and the accuracy of the predictions made by the revised model are statistically analyzed. The results show that these predictions are not significantly different from the field measurements, at a 95 percent confidence level.

When the original M/G/1 queuing model predictions are compared to the revised model predictions with a paired "t"-test, the results show the revised model predictions to have a significantly smaller mean difference from the field measurements than the predictions from the original model. Although both models are accurate when compared to the field data, the revised model is more accurate than the original model.

An interesting and unexpected result from this study involves an update of the Herbert capacity calculations.

These capacities are the current guidance used in the 1985 Highway Capacity Manual. The capacities are based on a straight line equation that was developed from the relationship of headway to traffic volume: H = 10.15 - 5*S, where S is the ratio of the traffic volume on the major street to the total intersection volume, and H is the average departure headway for through vehicles with loaded conditions. (Herbert, 1987) As volume split increases, headway decreases. The data collected in this study maintains this relationship, but the slope and intercept of the line change to: H = 7.7918 - 5.1306*S. It is derived in a similar manner to the method given in the Herbert study. This equation changes the basic capacities for the varying volume splits; the measured field capacities are greater than the capacities determined in the Herbert study.

The capacity values are different enough from the current Highway Capacity Manual guidance to warrant attention. Since this study is based on data from a limited geographic area, further study should be conducted before accepting this capacity data.

Four-Way Stops in the Field

The controversy surrounding the use of four-way stops is evident in and around the Columbus area. Some communities use four-way stops frequently while others do not seem to use them at all. The inconsistency of their use

prompted a series of interviews with practicing traffic engineers and other individuals who are involved in the decision-making process for sign installation.

I interviewed individuals in the Columbus area as well as a few traffic engineers in northern New Jersey and one in the City of Cleveland. I asked each person the same basic questions about their opinions or biases about the use of four-way stops, when they use them, when they should be avoided, and whether the signs are confusing to the average driver. I also asked the individuals if they knew of any legal cases involving the misuse of four-way stops.

The majority of people I interviewed work in the Columbus area. They all said that they have no particular bias for or against the use of four-way stops. Most people also feel that four-way stops are not confusing to the typical driver. They said that four-way stops should not be used as speed control devices. However, they can be beneficial for intersections that have equal traffic volume on both streets but not enough volume to warrant a traffic signal, particularly if there is an existing sight distance problem.

Everyone interviewed said that they use the Manual of Uniform Traffic Control Devices (MUTCD) as their guide for sign installation. However, it did not appear that everyone has the same standard for four-way stop installation as I drove through the different communities. I discovered that

the main reason for the differences are the political systems within the communities.

Many local residents request the installation of four-way stops in their neighborhoods as speed control devices. However, traffic engineering guidance cautions against this type of use because it leads to overuse of stop signs, and overuse fosters driver disregard for all stop signs. The four-way stops are also not an effective way to reduce overall speed.

The engineers are aware of the guidance, so when a request is made, they conduct a study to see if the sign is warranted. If it is not, they recommend disapproval of the request. Meanwhile, the citizen making the request has gone to their city council member for help with their situation, since the council has final authority over the approval or disapproval for requests involving traffic control devices. When the case is presented, the council usually overrides the engineering recommendation because they are more concerned about being re-elected than installing an unwarranted four-way stop. The council members' position is understandable, since there are so few cases brought to court concerning the illegal use of four-way stops.

The problem with politically installed signs is more of a problem in the smaller communities where the people in the community have greater access to the individuals in control. The City of Newark, Ohio has a tremendous problem with

political stop signs while the City of Columbus has virtually none of this type of problem.

Westerville, Ohio has found a good solution to this problem. They have a local ordinance requiring all traffic control devices to be installed in accordance with the Ohio MUTCD. This saves a lot of time and energy in the entire decision-making process and it also avoids the installation of illegal control devices.

The opinions of the individuals in and around Columbus do not necessarily coincide with the individuals I interviewed in New Jersey. All three New Jersey traffic engineers do not like four-way stops. They do not recommend their use and they feel that any intersection is controlled better either by a two-way stop or a traffic signal. This could be a regional difference. The people I interviewed work in the New York metropolitan area. Drivers tend to be more impatient so there is a real obedience problem. Also, traffic volumes are much greater, so it is probably easier to meet the warrants for signalization.

When I spoke with Cleveland's Chief Traffic Engineer I expected him to echo the opinions of the Columbus engineers, but he did not. He has similar beliefs as the New Jersey engineers. So perhaps the difference of opinion regarding the use of four-way stops has more to do with city size and driver tendencies than any other type of regional concern. One issue that everyone agreed upon is that four-way stops

should only be used if they meet the conditions given in the warrants. It is therefore important to ensure that the information in the guidance literature is accurate.

When stop signs are installed and they do not meet the warrants, there is the possibility of a lawsuit. A few suburbs of Cleveland have experienced lawsuits regarding the illegal installation of four-way stops. Some involved four-way stops that were installed as speed control devices. Others involved four-way stops that were installed for pedestrian protection. In these cases where illegal signs were installed, the courts ordered the cities to remove them.

CHAPTER I

THE HISTORY OF FOUR-WAY STOPS

1.1 Introduction

The use of four-way stops as a method of traffic control is not a new concept. We have found journals dating back to the late 1940s that discuss their use, advantages, and disadvantages. In fact, upon examining the development of the study of four-way stops at intersections, an interesting pattern appears. The research usually focuses on vehicle delay caused by the four-way stops, but the influence of current public concerns is also evident in the chronology.

1.2 The 1940s

The literature in the late 1940s indicates a growing use of four-way stops and a concern for their proper use. Harrison (1949) says that four-way stops are only justified in two situations: first, as a regulatory measure based on traffic volumes, and second, as a means to reduce angular-type accidents. He says that, "Observations show that large volumes of vehicular traffic can be handled with safety and reasonable efficiency under four-way stop control...[with] the greatest advantage to drivers and the least advantage to

pedestrians." This claim, which may have been valid in 1949, is much disputed in future literature.

Harrison goes on to say that the most common problem for state highway traffic engineers is the intersection of two State or US numbered highways. He believes that a series of warrants and values for the application of four-way stops need to be developed as a guide for traffic engineers. The ENO Foundation agrees; in 1950 they publish A Volume Warrant for Urban Stop Signs (Raff, 1950). In this book, the author discusses the vagueness of the then current Manual of Uniform Traffic Control Devices (MUTCD) with respect to stop sign warrants. He goes on to develop what he believes to be valid warrants based on detailed field observation, experimental results, and probability theory. The author also discusses application of the volume warrants. Unfortunately, he only considers two-way stops, so traffic engineers have no definitive warrants for four-way stops, and the subject is not formally discussed again until the late 1950s.

1.3 The 1950s

The literature in the early 1950s focuses on vehicle delay incurred at four-way stops and comparisons to other methods of traffic control. Hall (1953) compares the control device delay of an intersection that is first controlled by a four-way stop and subsequently converted to a semi-actuated signal. He finds both a greater delay per

vehicle and higher average total vehicle delay for the signal control over the stop sign control. His explanation of the results is based on the fact that four-way stops are in constant use, there is always a vehicle in the intersection, while traffic signals, by design, have periods of complete non-use (all-red intervals) during the cycle length.

Keneipp (1951) evaluates the efficiency of intersections when they are changed from two-way to four-way stop control. He finds that four-way stops are less efficient than two-way stops for all the intersections he studied; conversion to four-way control results in a time loss to the major street that is twice the time savings to the minor street. Keneipp believes that there is no logical warrant for a four-way stop except as a safety measure or as a device to satisfy pressure groups demanding action at an intersection that warrants no action. However, in future publications other traffic engineers caution strongly against the use of four-way stops for political purposes or to satisfy citizen pressure.

Many traffic engineers believe that using four-way stops to quell political pressure will result in a serious over-use of such control. Hanson (1957) says that overuse of stop signs causes unnecessary delay and creates driver disregard for all stop signs. Wilkie's study (1954) supports Hanson's claim of driver disobedience. Wilkie

found a general non-conformance to stops at high accident intersections, with a "serious percentage" of rolling stops and a "significant quantity" of no stops.

Hanson (1957) describes actions taken in Peoria, Illinois to try to curtail the stop sign disobedience problem. Peoria conducted a major stop sign removal program, which included a before and after study. Hanson gives a thorough discussion of the advantages and disadvantages of four-way stops. The advantages are: they control traffic where two-way stops are inadequate; they reduce accident rates at intersections that do not meet the warrants for signalized control; they are an appropriate transition from two-way to signalized control; and they have a tendency to reduce the number of right-angle accidents as well as the severity of all accidents. The disadvantages are: they cause a very large aggregate delay, representing a high economic loss when they are placed improperly; they may add congestion to an intersection, thereby increasing wear and tear on vehicles and fuel consumption; they create unreasonable delay when approach volumes and speeds are high, especially when there is more than one lane per approach; and, when they are used improperly, they tend to increase the number of accidents, especially rear-end collisions. Hanson therefore recommends some warrants for their use, based on vehicular volume and accident statistics.

The stop sign removal program in Peoria finally returns to the question of warrants for the use of four-way stops as a means of traffic control. In 1959, the warrants suggested in Hanson's program were studied and subsequently accepted by the Institute of Traffic Engineers (ITE). In order to justify the use of a four-way stop, at least one of the following warrants must be met:

(1) Volume Warrant: Minimum vehicular volume in urban areas must have a total volume greater than 500 vph for any six hours of an average day with a minimum of 35% of the vehicles entering from the minor street. The minimum vehicular volume in rural areas must show a total volume greater than 400 vph for any six hours of an average day with a minimum of 35% of the vehicular volume entering from the minor street. When the total volume for either rural or urban areas exceeds 1000 vph for any six hours of an average day and the minor street has more than 250 vph for the same six hours, installation of a traffic signal should be considered.

(2) Accident Warrant: There must be five or more reported accidents with a value greater than $100 (injury or property damage), of a type correctable by four-way stops, or lesser measures (including two-way stops) have failed to improve the accident record.

In addition to establishing warrants, the ITE committee discusses specific misapplications of four-way stop control

that are to be avoided. They claim that much of the disrepute associated with four-way stops is caused by inappropriate use. Four-way stops are not to be installed (1) within 1000 feet of a signalized intersection or within a system of coordinated traffic signals on a thoroughfare, (2) as a result of public pressure that comes about from a spectacular or much publicized accident, (3) as a speed reduction device, and (4) as a cure-all for speed or school crossing protection problem (to aid pedestrian crossing). ITE again emphasizes that improper use encourages violation and generates serious disregard for stop sign control in general.

The ITE also states that future research is needed for four-way stops. The control warrants are based on volume and accident rates, but little research exists to substantiate their validity. There is a need for a comprehensive study of all types of intersection control devices, with the subsequent development of a significant index of effectiveness for each type of traffic control.

There is still much controversy over the use of four-way stops. Some traffic engineers believe they should be used more frequently while others believe there is no logical warrant to justify their use.

1.4 The 1960s

The next documented study of four-way stops is conducted in 1963 by Jaques Herbert (1963); it involves the

determination of capacities of four-way stop intersections under various traffic and operating conditions, based on departure headway. He finds that (1) variations in the split of traffic volume on the major and minor approaches produce significantly different headways of departure; (2) left-turning vehicles have no effect on capacity; (3) for each 1% of right-turning vehicles, the capacity of the intersection is increased by 0.2% ; (4) under pressurized and ideal traffic conditions, through passenger cars per lane can be expected to discharge at a rate of one every 7.65 seconds (with a 50/50 traffic volume split) or one every 7.15 seconds (with a 60/40 split), and the discharge rate will become one vehicle every 4.05 seconds if the split is 100/0; and (5) 70% of the vehicles will move two-abreast if there are two lanes on a loaded approach.

Herbert used his results to determine basic intersection capacity (passenger cars per hour) for various traffic splits. The capacities are shown in the following table:

Table 1: Basic Capacity of Intersections for Various Traffic Splits (Herbert, 1963)

SPLIT	BASIC CAPACITY (vph)
50 / 50	1900
55 / 45	1800
60 / 40	1700
65 / 35	1600
70 / 30	1550

These values were subsequently adopted in the 1965 Highway Capacity Manual and are still being used today in the 1985 Highway Capacity Manual.

1.5 The 1970s

For the next fifteen years there is no attention given to four-way stops in the literature; a status quo seems apparent. There is one study during this time period, in 1971, where Haenel, Lee, and Vodrazka (1971) examine traffic delay and warrants for control devices through use of the digital delay recorder. They discover a reduction in average delay for stopped vehicles and an increase in total delay per intersection when control is changed from two-way to four-way stops. Their study denotes the advent of the computer age in traffic engineering.

The energy crisis of the mid 1970s once again brings attention to four-way stops in the late 1970s and into the 1980s. Now, in addition to delay, researchers consider fuel consumption and vehicle emissions in their studies.

In 1978, Hall, Michael, and Sinha (1978) examine non-signalized control at low volume intersections. Their study considers the influence of intersection conditions on safety, travel time, fuel economy, and exhaust emissions. Based on the data, they show yield signs to be the most desireable type of control at low-volume intersections because they optimize the trade-offs between the factors studied.

In 1979, Lee and Savur (1979) developed the TEXAS (traffic experimental and analytical simulation) model for unsignalized intersections. It is a computer program that enables traffic engineers to determine either level of service, intersection geometry, type of control or volume of traffic accommodated when the other three factors are known.

1.6 The 1980s

In 1982, Briglia (1982) gets back to the question of delay and the associated costs at four-way stop intersections. He evaluates accident experience, motor vehicle operating costs, travel time, fuel consumption, and air quality impacts at low-volume, high speed rural intersections that are controlled by four-way stops. His conclusion qualitatively states that installation of four-way stop control for the purpose of decreasing accidents is accompanied by a substantial increase in other costs.

Traffic engineers once again examine the basic need for four-way stop control at intersections in the 1980s. Carter and Chadda (1983) summarize the use of four-way stops over the past few decades. The reasons they cite for installation are (1) conformance with MUTCD warrants, (2) as an interim measure before installing traffic signals, (3) as a safety improvement at intersections with inadequate sight distance, (4) as a speed control device, and (5) city officials yielding to public pressure to take action when none is warranted. The impact of the installation is an

increase in stops; this results in delays, an increase in vehicle operating costs, and an increase in emission of pollutants. Unwarranted stops breed disrespect for all stop signs as well as affiliated safety, economic, operational and environmental problems.

By this time, jurisdiction officials have a better understanding of the costs associated with four-way stops. They question the need for stop signs at many locations and several stop sign removal programs are initiated. The programs differ greatly in method and procedure. In an attempt to take advantage of the successes and failures of the different programs, Carter and Chadda (1983) develop a standardized process for stop sign removal. Their removal program requires public awareness in order for it to be successful. Here are the steps they recommend: (1) inventory all multi-way stops in the jurisdiction; (2) determine whether MUTCD warrants are met; (3) prioritize the intersections needing stop sign removal; (4) prepare a detailed analysis of each site; (5) identify and quantify the benefits of removal; (6) identify locations for less-restrictive control; (7) make recommendations to elected officials and decision makers; and (8) involve citizens and community groups to help publicize the changes. In 1985, Chadda and Mulinazzi (1985) reemphasize the need for a standardized stop sign removal program. Their procedure is basically the same as Carter and Chadda's, but they include

more details. They recommend the use of advance warning signs (with an acclimatization period of 30 to 90 days after the removal) and either simultaneous or staggered removal of the unnecessary signs, depending on the size of the target area. They also suggest making follow-up studies to evaluate driver attitudes, safety problems, and approach volume and speed data. Stop sign removal programs should now be fairly standardized and successful.

Meanwhile, traffic engineers are still studying the necessity of four-way stops. In addition to Briglia's study (1982), which found four-way stops to be a cost-effective method of accident reduction at rural Michigan intersections, Byrd and Stafford (1984) determine that four-way stops should not be used at South Carolina's low-volume, low-speed intersections unless an accident problem exists, due to the "unnecessary delay and user costs." The controversy continues.

In an effort to clear up some of the controversy, Richardson (1987) develops an analytical delay model to improve upon the volume split and capacity given in the 1985 Highway Capacity Manual. The delay model uses queuing theory to predict delays; it shows good agreement in terms of capacities and levels of service for previously reported demand splits. The model adds the ability to predict levels of performance over a much wider range of operating conditions.

The most recent study dealing with four-way stops was conducted in San Diego, California. (Celniker, 1988) The city engineers are not happy with the warrants for installation in the MUTCD. They do not feel that the warrants cover all the issues. Intersections with "unusual conditions" (such as limited sight distance) or mixed situation (such as moderate volumes and a few accidents) do not warrant four-way stops according to current guidance. However, from a safety standpoint, these intersections might benefit from a four-way stop. So the city tested a policy where intersections are scored on a point basis. Points are given for accident experience, unusual conditions, traffic volumes, traffic volume difference, and pedestrian volumes. If the intersection scores a certain number of points, then a four-way stop is justified, regardless of what the current warrants dictate.

The city tested this policy by doing a before-and-after study at intersections that were recently changed to four-way stops, but they did not meet MUTCD warrants. The study shows that intersections that meet the new policy point requirements had a 64 percent reduction in accidents while intersections that do not meet the point requirements had no reduction. (Celniker, 1988) The hope of the San Diego engineers is to establish new warrants based on this system. They say that it will give engineers more confidence when making decisions and it will help improve safety. This is a

very new study, so it will probably take some time before we see any impact in the field.

Celniker's work is the most recent in the development of the study of four-way stops as a method of traffic control. The evolution started with an emphasis on warrants, continued with a focus on capacity, then, with the onset of the energy crisis, traffic engineers became concerned with fuel and emissions in addition to delay costs. There was also the problem of political pressure to install unwarranted stop signs to placate pressure groups. Now the focus is back to capacity and level of service. Throughout the history, there is much controversy over the use of four-way stop control and traffic engineers still cannot agree. The solution seems to be that there are both good and bad factors associated with the use of four-way stops; practicing traffic engineers must be aware of all of them, and install four-way stops only when they are warranted.

CHAPTER II

VALIDATION OF THE RICHARDSON M/G/1 QUEUING MODEL

2.1 Purpose

The at-grade urban intersection is one of the most important concerns for traffic engineers. Approximately one half of all urban accidents and more than three fourths of all urban delays are related to urban intersections (Herbert, 1963). Traffic engineers want to provide safe and efficient movement through intersections for both vehicular and pedestrian traffic alike. They do this by regulating movements by control devices. One type of control is the four-way stop.

The four-way stop is a very controversial traffic control device. Despite the controversy, it is not a very well-researched method of traffic control. In 1987, the Transportation Research Board identified a need for further study of four-way stops in the areas of delay, capacity, and level of service (TRB, 1987).

Chapter 10 of the 1985 Highway Capacity Manual contains a small section on multi-way stops (HCM, 1985). It has a table that enables the traffic engineer to estimate intersection capacity based on the volume of the major and

minor roads of the intersection. This table is unchanged from the 1965 Highway Capacity Manual. It is based on a study conducted in 1963 by Jacques Herbert. Herbert studied three intersections and measured average departure headways; from the data he collected, he determined the intersection capacity. (Herbert, 1987)

In 1987, Anthony J. Richardson developed a model to try to improve upon Herbert's capacity table. (Richardson, 1987) He used a queuing model to predict average time in the system. The only information required by the model is the traffic volume and the number of lanes in each approach of the intersection; other necessary information is predicted by a second part of the model. Richardson took departure headway values from Herbert's study and incorporated them into the second part of the model which involves the prediction of the service time. Richardson compared his results to the 1985 Highway Capacity Manual and to a simulation model and his results show good agreement. However, he does not support the model with a field study.

The purpose of this research is to see how well the Richardson model predicts measured data that occurs in the field. Included in this is an evaluation of whether the departure headway values taken from the Herbert study heve changed over the past twenty five years, since these values are an integral part of the model prediction process. If the departure headways are significantly different, this

study will determine how the Richardson M/G/1 queuing model is affected.

If the field data supports Richardson's model predictions, then there will be strong evidence to show that this model is an improvement upon the information in the current Highway Capacity Manual. The model allows the traffic engineer to predict intersection capacity using only traffic volumes and number of approach lanes. This model has the accuracy of a simulation model, but it does not require the same amount of detailed information.

2.2 1963 Herbert Study Background

(The *information* in Section 2.2 is from Herbert, 1987.)

2.2.1 Purpose

In 1963, Jacques Herbert conducted a study to determine the capacities of four-way stop controlled intersections under various operating conditions. He derived capacities from the average departure headways measured as vehicles entered the intersection.

2.2.2 Scope

The Highway Capacity Manual defines basic capacity as the "maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained." (Herbert, 1963) To satisfy this

definition, Herbert collected separate departure headway values for passenger cars, commercial vehicles, left- and right-turning vehicles, and through vehicles. He then calculated the possible capacity, which is the capacity under prevailing traffic and road conditions, by applying a reduction factor of 0.8 to the basic capacity. This reduction factor was taken from the Highway Capacity Manual.

Another definition of practical capacity of an intersection is based on delay criterion. Assuming that there is a random distribution of vehicles on an approach, the departure headway can be used to determine the vehicular volume that will cause a certain percentage of the drivers to be delayed a preferred amount of time. Any volume that causes a greater percentage of drivers to be delayed by the same amount of time, or causes the same percentage of drivers a greater time delay, is above the practical capacity. Herbert derives a set of curves for various time periods and percent of drivers based on the measured average departure headway.

Herbert observed three intersections in the Metropolitan Chicago area. He used a movie-camera technique to simultaneously film all approaches of each intersection for 80 minutes. From the film he measured the departure headways. He also determined how left- and right-turning vehicles, number of lanes on the cross-street, and the volume split between the intersecting streets affects

capacity.

2.2.3 Locations Studied

Three intersections in Metropolitan Chicago were studied.

INTERSECTION A: Willow and Hibbard

This intersection is located in Winnetka, a community in the outlying northern Chicago suburbs. This intersection was selected because of its high volume, the nearly equal volume split, and its high percent of turns. It also had very little pedestrian and roadside interference. All sight distances are adequate and there is one lane per approach in all four directions.

INTERSECTION B: Winnetka and Hibbard

This intersection is located just south of Intersection A, in the same community of Winnetka. Its traffic volume is not as high. It was selected for study because of its difference in volume split. Like Intersection A, it had a high number of vehicle turning movements and almost no interference from pedestrians, parking, or driveways. There is one lane per approach in all four directions.

INTERSECTION C: Cumberland and Devon

This intersection is very different from the first two: it is a two-lane road crossing a four-lane road, and the two-lane road has parking on both sides. It is located in Park Ridge, a well-developed community, so there is some interference from pedestrian movement and parking.

2.2.4 Tests on Data

It is necessary to measure the headway of departure of vehicles on the intersection approach during loaded conditions in order to calculate the basic capacity of four-way stop intersections. If both streets are loaded the vehicles will proceed through the intersection in turn, each moving into position as a cross-street vehicle accelerates through the intersection. Frequently, intersection conditions exist where there are vehicles queued on one approach while there are few or none on the cross-street. In this case, one would expect the headways to be smaller on the street with no vehicles on the cross-street, since they do not have to wait as long. For this reason, Herbert categorized headways by the type of intersection load and he recorded separate values for passenger cars, commercial vehicles, left- and right-turning vehciles, and through vehicles.

The headway types are classified as follows:

L HEADWAYS: When both streets are loaded vehicles proceed through in turns, with one vehicle accelerating from a cross-street approach within the headway recorded.

N HEADWAYS: The approach under study is loaded with no vehicles waiting at the stop line or approaching on the cross-street (50 feet or less).

I HEADWAYS: The approach under study is loaded, with interference from vehicles on the cross-street (within 50

feet of the stop line). Vehicles on the cross-street do not proceed into the intersection before vehicles on the approach under study, they merely cause interference and hesitation.

A total of 321 passenger car headways was recorded for Intersection A, 324 for Intersection B, and 210 for the lane studied at Intersection C. The data collected for the commercial vehicles was negligible.

Two statistical significance tests were run on the data at a 95 percent confidence level. First, the normal distribution test (two-sided), and second, the student's "t" distribution test (two-sided). Both tests give the statistical significance of the difference in means of two sets of data. The "t" test is preferable when working with samples that have fewer than 30 values because it does not require population values.

The results show that left turns have no effect on departure headway but right turns do have an effect. The volume split has no effect on total intersection type N or type I headways, but it does affect total intersection type L headways. This difference could be attributed to intersection location, geometric configuration, or sight distance at the intersection. The three types of headways (L, N, I) were significantly different. Also, significantly longer headways were needed to cross a four-lane street rather than a two-lane street.

This statistical analysis formed the basis that Herbert used to estimate the various factors affecting the traffic behavior at intersections.

2.2.5 Presentation and Discussion of Results

Table 2 shows the headways of departure of passenger cars for the conditions shown, included also are the standard deviations for each sample group. These values are used for comparison of today's field measurements to see if there is a difference in departure headway.

2.2.5.1 FACTORS AFFECTING HEADWAY:

Split: The results show significantly different type L departure headways for different volume splits. Since type L headway is used in calculation of capacity, Herbert had to account for this difference. He assumed split is the most influential factor and derived an equation of headway as a function of split. Since two splits were available, he got the following straight line equation:

$$H = 10.15 - 5S \tag{6}$$

where H is the average departure headway for through passenger vehicles under loaded conditions and S is the ratio of the volume on the major street to the volume of the total intersection. As split increases, headway decreases and a larger volume can be handled on the major approach.

Left Turns: Under loaded conditions, left-turning vehicles did not take significantly longer than through

vehicles. He concluded that left-turning vehicles have a negligible effect on headway and therefore on capacity also for four-way stop intersections.

Right Turns: Herbert observed that right-turning vehicles have significantly lower headways and consequently contribute to an increase in capacity.

Commercial Vehicles: Data collected on commercial vehicles was negligible, so no conclusions could be drawn.

2.2.5.2 CAPACITIES:

Basic Capacity: is the maximum number of passenger cars that can be handled in one hour, under the most ideal conditions. The intersection must be loaded with queues of vehicles waiting on all approaches.

For a two-lane street versus a two-lane street intersection, maximum volumes will be handled when vehicles on opposite approaches accelerate simultaneously, alternating with cross-street vehicles. This is idealistic; actual observed performance is somewhat different. Under ideal conditions, an equal traffic volume will be handled on both streets, resulting in a 50/50 volume split. Herbert calculated the average headway for this case from Equation 1 with S = 0.50, and found the headway value to be H = 7.65. He then calculated the basic capacity as (3600sec/hour)/(7.65sec/veh)x(4 approaches) = 1885 or about 1900 passenger cars per hour.

Herbert also noted that if there is no traffic on the cross-street (100/0 volume split), the headway becomes 4.05 seconds and the intersection capacity is approximately 1800 passenger cars per hour. He concluded that the maximum number of 900 passenger cars can be handled on each approach at a two-way stop with no cross-flow traffic and a continuous load. Any other conditions will decrease capacity.

Herbert calculated the basic capacity for various traffic volume splits (Table 2) with the following equation:

$$\begin{array}{c}\text{Total}\\ \text{intersection}\\ \text{basic capacity}\end{array} = \begin{array}{c}\text{volume on}\\ \text{loaded street}\end{array} + \begin{array}{c}\text{volume on}\\ \text{other street}\end{array}$$

$$= \frac{(3600)}{(10.15-5S)} \times 2 + \frac{(3600)}{(10.15-5S)} \times 2 \times \frac{(1-S)}{S}$$

$$= \frac{(7200)}{(10.15-5S)S} \qquad (7)$$

Possible Capacity: applies adjustment factors to the basic capacity based on turning movements, interference (such as pedestrian movement or parking), and commercial vehicle traffic. Based on his results, Herbert makes no adjustment for left turns. He increases capacity by 0.2 percent for each 1 percent of right turns in the total traffic volume. He suggests a 0.9 reduction factor for interference, although he does not have the statistical data to support this. Finally, he recommends reducing capacity by 1 percent for each 1 percent of commercial vehicles in the total traffic volume.

Practical Capacity: is about 80% of the possible capacity at a signalized intersection, according to the Highway Capacity Manual. Herbert applies this factor to four-way stops to determine practical capacity (Table 3).

2.2.6 Poisson Distribution Applied to Delays and Practical Capacity

Delay is the best single judge of intersection capacity because it causes the most inconveniences to the driver. The Highway Capacity Manual definition of practical capacity is based on delay. Assuming random distribution of arrival vehicles with a Poisson distribution, Herbert computes the volume of traffic that causes drivers to wait a certain amount of time. Herbert used this information to develop a set of curves that show how mean traffic volumes affect the percent intersection clearance for periods of time ranging from 20 to 50 seconds. He estimates the maximum acceptable waiting length at a four-way stop to be about 30 seconds. This means that at 90 percent clearance, 319 vehicles will go through the intersection in one hour with an average headway of approximately 6 seconds.

2.2.7 Conclusions

At the close of this study, Herbert notes the following findings:

1. Variations in the split of volume between the two intersecting streets of a four-way stop intersection produce a significantly different headway of departure for two

different intersections.

2. Left-turning vehicles have no effect on the capacity, the average headway of departure for left-turning and through vehicles not being significantly different under various traffic conditons.

3. For each 1 percent of right-turning vehicles, the capacity is increased 0.2 percent.

4. Under pressurized and ideal traffic conditions through passenger cars per lane may be expected to be discharged across a two-lane road at an average of one every 7.65 seconds if the split is 50/50, and one every 7.15 seconds if it is 60/40. These rates are averages for the whole intersection.

5. If the split becomes 100/0 (i.e., all on-coming vehicles are on two opposite approaches only) and for the same conditions as in item 4, one might expect a discharge rate of one vehicle every 4.05 seconds from each of the two approaches.

6. For the conditions of item 4 and a 50/50 split, the capacity per lane averages one vehicle every 8.08 seconds if the street to be crossed has four moving lanes.

7. Seventy percent of vehicles are found to be moving two abreast if there are two lanes on a loaded approach.

Some of this information is taken and used by Anthony J. Richardson in his development of the M/G/1 queuing model.

2.3 Development of the M/G/1 Queuing Model

(The information in Section 2.3 is from Richardson, 1987.)

2.3.1 Purpose

There are a limited number of empirical studies and simulation models of capacity and delay at multi-way stops available in published literature. However, there are no available analytical models of delay at multi-way stops. Richardson therefore undertakes a study to develop such a model. He uses data from the 1963 Herbert study as input parameters to an M/G/1 queuing model which predicts delays at four-way stops.

2.3.2 Development of the Model

Richardson chooses to develop an analytical model based on the concept of queuing theory rather than a discrete event digital simulation model because of the greater length of computation time required by a simulation model.

Richardson uses an M/G/1 model with negative exponential arrival rates, general distribution of service rates, and a single server (the intersection) as the basic queuing model. This model assumes a random arrival rate, which is a likely condition at four-way stops unless the traffic flows are high. With high traffic flows, the assumption of random arrival may not be valid and the delay results predicted by the model may not be accurate.

The single server is the intersection, but the queuing discipline is somewhat unusual since vehicles are processed through the intersection by a form of priority queuing. Priority is assigned to the vehicle waiting longest at the stop line, but not necessarily the longest in the system. The process is further complicated since it is possible for the intersection to serve two vehicles simultaneously if they arrive on non-conflicting approaches. No existing general queuing model could accommodate this, so Richardson develops a specific model for the multi-way stop.

The M/G/1 queuing model involves the Pollaczek-Khintchine formula, which says:

$$L = [2P - P^2 + \lambda^2 V(S)] / [2(1 - P)] \qquad (8)$$

where

- L = average number in the system (average number on the approach, including the vehicle at the stopline);
- λ = average arrival rate;
- S = average service time (Herbert's departure headway);
- $V(S)$ = variance of service time; and
- P = utilization ratio (= arrival rate*service time).

Using Little's equation, the average time in the system is then determined by

$$Ws = L / \lambda \qquad (9)$$

where Ws is the average time in the system. The major problem now is to find the average service time for the intersection and its variance. Richardson derives some equations based on vehicle movements through a four-way stop.

2.3.3 <u>Derivation of Equations</u>

To illustrate the basic concept behind Richardson's calculation of service times, he first considers the simple four-way stop intersection, where flow exists only on the northbound and westbound approaches, and all vehicles are proceeding straight through the intersection. Considereing a vehicle that arrives on the northbound approach, Richardson then calculates its service time, where he defines service time as the time between this vehicle's departure and the time at which a vehicle immediately in front could have departed. If there is no vehicle waiting on the westbound approach, then this vehicle can follow the previous northbound vehicle through the intersection at the minimum allowable headway, t(m) . Richardson takes the value of 4.0 from Herbert's study as this minimum allowable headway. However, if there is a vehicle waiting at the westbound approach when the northbound vehicle arrives at the stop line, the northbound vehicle must wait for the westbound vehicle to clear the intersection before it can proceed. In turn, the westbound vehicle must have waited for the previous northbound vehicle to clear the intersection. The intersection clearance times are taken from Herbert's study as t(c) = 3.6 + 0.1(number of crossflow approach lanes from both directions). So for a simple four-way stop with one lane on each approach, t(c) is 3.8, since there are two cross-flow lanes. The total clearance time

T(c) is the sum of the clearance times on each approach, and it is equal to the service time for a northbound vehicle which arrives when a westbound vehicle is waiting at the stop line.

The average service time for a northbound vehicle is then given by

$$Sn = t(m) * (\text{probability of no westbound vehicle at the stop line}) + T(c) * (\text{probability of a westbound vehicle at the stop line}) \quad (10)$$

The probability of a westbound vehicle being at the stop line when a northbound vehicle arrives depends upon the utilization ratio of the westbound approach. The utilization ratio is the probability that the system is not empty at any point, so

$$Sn = t(m) * [1 - Pw] + T(c) * Pw \quad (11)$$

The average service time on the northbound approach is therefore a function of the average service time on the westbound approach. By symmetry, the average service time on the westbound approach is a function of the average service time on the northbound approach, where

$$Sw = t(m) * [1 - Pn] + T(c) * Pn \quad (12)$$

Substitution of Equation 5 into Equation 4 makes it possible to solve directly for Sw and Sn and then to proceed with the analysis to obtain the delays on each approach. However, the situation becomes more complex when there are multiple conflicting flows at the intersection.

For the more general situation of flow on all approaches of a four-way stop, the same basic equations Richardson outlined still apply. However, the utilization ratio must apply to the east-west approach as a whole when determining the average service time on the northbound approach, since the northbound traffic must yield to vehicles waiting on either the westbound or the eastbound approaches. Therefore,

$$Pew = 1 - (1 - Pe)*(1 - Pw) \quad (13)$$

By similar reasoning,

$$Pns = 1 - (1 - Pn)*(1 - Ps) \quad (14)$$

The average service times on both the northbound and southbound approaches will be functions of the flows and service rates on both the eastbound and westbound approaches. From this, Richardson concludes that the service time on the northbound approach will be the same as the service time on the southbound approach since they must yield to the same east and westbound traffic. Similarly, the service times on the eastbound and westbound approaches will be functions of the northbound and southbound flows and service times. These interactions yield a series of equations that are mathematically intractable in a closed-form solution.

The problem becomes more complicated when multiple lanes on each approach is considered. For this situation, the utilization ratios are given by the following equations

if the drivers on each approach split equally between the available lanes.

$$Pew = 1 - (1 - Pe/L)^{L} * (1 - Pw/L)^{L} \tag{15}$$

$$Pns = 1 - (1 - Pn/L)^{L} * (1 - Ps/L)^{L} \tag{16}$$

where L is the number of lanes on the appropriate approach.

Since these equations do not give a closed-form solution to the determination of average service times, Richardson adopts an iterative approach to obtain stable values of service times on the four approaches. He first assumes initial values of the service time on each approach, calculates the utilization ratios, and then substitutes the approach utilization ratios into Equations 8 and 9 to get the effective blocking utilization ratios for each approach. The blocking utilization ratio is, for example, the utilization ratio for the east-west approach as perceived by a northbound driver as he is blocked from proceeding through the intersection by either an eastbound or westbound vehicle at the stop line. Richardson then substitutes the blocking utilization ratios into Equations 4 and 5 to get updated values of the average service time on each approach. This procedure is iterated until equilibrium is reached. The initial assumed values of the service times are bounded by t(m) and T(c) because these are the service times at zero and maximum conflicting flow, respectively.

Once Richardson calculates the average service rates, calculation of the variance of the service rates is fairly straightforward. Since service time distribution is bimodal (only values of t(m) and T(c) are possible), he calculates the variance as follows:

$$V(S) = t(m)^2 * [T(c) - S] / [T(c) - t(m)] + T(c)^2 * [S - t(m)] / [T(c)-t(m)] - S^2 \qquad (17)$$

Richardson is now able to use these values for the average and variance of the service times in Equations 1 and 2 to predict the delay on each approach.

The delay from acceleration and deceleration time is a significant part of total delay for an intersection with low flow conditions. This total delay can be calculated by estimating the time it takes for a vehicle to decelerate and the time it takes for the vehicle to accelerate back up to the posted speed limit, and add these values to the predicted delay from Richardson's model.

2.3.4 Application of the Model

This model enables the user to predict delay and estimates of average queue length on each approach of an intersection using only the number of lanes and the total flow for each approach. The model does not consider turning movements. When Richardson published his M/G/1 queuing model, he Richardson was currently working on a model that incorporates turn movement data.

The model can be used within an equilibrium assignment network program. This is done by identifying the links in the network that represent conflicting approaches for the approach in question. Flows are read and delays can then be calculated.

Richardson applies the model to a range of general situations to show the effects of changing the approach flow, the conflicting flows, the flow from the opposite direction, and the number of lanes on the approach. From these results, he developed a set of curves that show system delay as a function of approach flow and conflicting flow for a two-lane versus two-lane intersection. System delay is defined as the time from when a vehicle joins the queue until that vehicle leaves the stop line. He finds that at zero east-west flow the capacity of the northbound approach is 900 vph. This agrees with Herbert's conclusion for the 100/0 volume split. The model results also agree with Herbert's 1900 vph capacity for the 50/50 volume split, with values of 475 vph flows for each approach.

Richardson then takes the results from the system delay curve calculations and creates a capacity table similar to the table in the 1985 Highway Capacity Manual (Table 4). There is approximate agreement between the capacities from the 1985 Highway Capacity Manual and the capacities predicted by the delay model for the demand splits that are common to both. Richardson notes that minimum intersection

capacity occurs with an 80/20 split, so the multi-way stop works best when there is balanced flow on both approaches or when there is no flow on one approach.

In varying the input data, Richardson finds that changing the number of approach lanes affects system delay. As expected, capacities are increased and delays are decreased when lanes are added to the intersection approach. He also finds that increasing southbound flow causes a marginal increase in the delays suffered by the northbound vehicles. The reason for this is as southbound flow increases there is less opportunity for east-west traffic to cross the intersection, thereby increasing the east-west queues. However, this tertiary effect is negligible on the overall northbound delay and can be ignored in most cases, particularly where the conflicting flows are either very low or very high.

2.3.5 Validation of the Model

Richardson validates his model by comparing its predicted delay results to the 1985 Highway Capacity Manual and to Lee and Savur's TEXAS simulation model. He does not validate the model with field data.

Average delay for Level of Service C is generally recommended as 30 seconds. Richardson compares his model's predicted delays to the flow and demand split at Level of Service C that are given in the 1985 Highway Capacity Manual. The model results are generally consistent with the

30 second standard average delay (Table 5).

Richardson also compares results from the delay model with the flow-delay curve from the TEXAS simulation model that was developed by Lee and Savur (1979). The flow-delay curve shows average system delay as a function of the total flow entering the intersection. Richardson superimposes the results from the delay model over the TEXAS simulation model flow-delay curve, and the result is quite good. The results predicted by the delay model are very consistent with those produced by the TEXAS simulation model.

Richardson's delay model has an advantage over the TEXAS simulation model. It has the added ability to predict levels of performance over a much wider range of operating conditions without the need for detailed simulations.

2.3.6 Conclusion

The delay model that Richardson developed can be used to estimate the delay characteristics of a specific multi-way stop intersection. It can also be used as a subroutine to calculate delays at multi-way stops within the framework of a network assignment model. The model results are substantiated by comparison with existing data in the 1985 Highway Capacity Manual and with results from the TEXAS simulation model.

2.4 Methodology

The validation of the Richardson M/G/1 queuing model consists of three basic parts. First is the validation of the departure headway values taken from the 1963 Herbert study. Second is the validation of the Pollaczek-Khintchine formula, which is used within the M/G/1 queuing model. Third is the validation of the average and the variance of service time predictions that are made by the model. Since different departure headways are expected, the effect of these different values on the service time and its variance will also be examined.

Field data measurements are used for all three parts of the validation process. The departure headways are measured in the field then compared to the values in the Herbert study to see if any difference between the two is statistically significant. The headways are measured by the type of movement through the intersection (left turn, through, or right turn), and by the type of intersection load. The type of intersection loads are the same as the ones measured in the Herbert study; they are separated into the following three categories (Herbert, 1987):

1. L headways: When both streets are loaded vehicles proceed through in turns, with one vehicle accelerating from a cross-street approach within the headway recorded.

2. N headways: The approach under study is loaded, with no vehicles approaching on the cross-street (within 50

feet or less) or waiting at the stop line.

3. I headways: The approach under study is loaded, with interference from vehicles on the cross-street (within 50 feet of the stop line). A vehicle on the cross-street does not enter the intersection before the vehicle on the approach under study, it merely causes interference and hesitation to the driver on the approach.

Field data measurements are used to validate the M/G/1 queuing model. In order to do this, measurements are made of all components of the equations used within the model. These are total intersection volume count, departure headway (separated by load type and turn movement), average arrival rate, and average time in the system. The variance of the departure headway (or service time) is not measured; it is a statistical calculation based on the measured service times.

Herbert's definition of departure headway is the same as Richardson's definition of service time, which is "the time between this vehicles departure time and the time at which a vehicle immediately in front could have departed." (Richardson, 1987). The definition of system delay is the time elapsed from when a vehicle joins the queue until it leaves the stop line. For the remainder of this study, the service time and system delay terms are used according to these definitions.

Once the data is collected, a statistical analysis is conducted to see if there is a significant difference

between the Herbert service times and the field data, and between the model's predicted system delays and the field system delays.

Comparison of the model predictions is in two parts. The first is predictions made by the Pollaczek-Khintchine formula using the field data for the average and variance of the service times. The second is predictions made by the Pollaczek-Khintchine formula using the model predictions for the average and variance of the service times. The reason for this separation is to enable the analysis of the accuracy of the two parts of the model.

2.4.1 Intersection Location

The intersections that are analyzed in this study were selected mainly on the basis of their traffic volume and the volume splits. *Diagrams of each intersection are given in* Figures 1, 2, and 3.

INTERSECTION 1: Park Road and Worthington-Galena Road

This intersection is located in the City of Columbus, northeast of the City of Worthington. It experiences heavy use, especially by commuter traffic. In the recent past, this was an intersection of typical country roads. However, there has been rapid development in the area in the past few years. Apartment complexes, housing developments, and convenience-type shopping areas now occupy what used to be fields. The roads have not been improved to meet the increase in demand, and this intersection is notorious for

excessive delay at rush hour. The intersection has virtually no pedestrian traffic and a fairly small percent of commercial vehicle traffic.

INTERSECTION 2: West New England Avenue and Evening Street

This intersection is located in the City of Worthington, a northern suburb of downtown Columbus, Ohio. Worthington is a well-established community. This intersection is in a residential area that is south of one highly traveled primary road and west of another highly traveled primary road. It experiences a significant level of cut-through traffic, vehicles driving through the neighborhood in an effort to avoid the traffic signals on the primary roads. There is minimal pedestrian traffic and negligible commercial vehicle traffic.

INTERSECTION 3: West New England Avenue and Oxford Street

This intersection is located one block east of Intersection 2, also in Worthington. It has very similar conditions: it is in a residential area with negligible pedestrian and commercial vehicle traffic. The main reason for its selection is its similarity to Intersection B in the Herbert study. The intersections are very close in volume split and percent turns. It therefore provides a good basis for comparison of any difference in service times.

2.4.2 Determination of Minimum Sample Size

The sample size required of a given data set to yield a particular range of accuracy is dependent upon the desired

confidence level and the standard deviation of the population. If no data is available, the standard deviation can be assumed. If, however, some data is available, an estimate of the standard deviation can be made from that sample. It is then possible to determine the minimum sample size required for a given level of confidence.

During the intersection selection process, I collected sample service times for the three types of intersection loads (L, N, and I). This information is then used in the following equations to calculate sample sizes required for a 95 percent level of confidence:

sample standard deviation:

$$s = \frac{\sum (x_i - \bar{x})^2}{n - 1} \qquad (18)$$

where

x_i = each data point in the sample,
$\bar{x}$ = sample mean,
n = sample size (number of data points), and
s = sample standard deviation.

required sample size:

$$N = \frac{1.96*s}{B}^2 \qquad (19)$$

where

s = sample standard deviation,
B = specified error of estimation,
N = sample size required, and
1.96 = z-value for 95 percent confidence level.

The specified error estimation, B, comes from the accuracy of the measurements. An electronic stopwatch, accurate to 0.01 seconds was used to measure the service

times. Of course, human error needs to be considered, so minimum sample size requirements are calculated for a stop watch measurement accuracy within 0.5, 0.25, and 0.1 seconds. The results of these calculations are given in Table 6.

It is reasonable to expect an accuracy of 0.5 seconds, so minimum sample sizes for load type L, type N, and type I are 21, 28, and 19, respectively. Actual field collection yielded much larger sample sizes, so the results are consistent with a greater accuracy at a 95 percent confidence level.

2.4.3 Limitations of the Study

This study was conducted at three intersections in outlying northerns areas of metropolitan Columbus, Ohio. Since the intersections are in the same general area, similar driver behavior can be anticipated.

All data was collected during the day and during similar weather conditions. There were dry roads and clear, sunny skies for all days during the data collection. Night and/or wet weather conditions are not investigated.

The data was collected at the same time each day on four different days, one day for each approach of the intersection. Data collection days were Tuesdays and Wednesdays, and one on a Thursday. This was an attempt to eliminate day of the week variations in traffic patterns and behavior, thereby enabling the assumption of similar volume

and traffic types over the collection period. Data collection dates and times for all approaches of each intersection are given in Table 7.

2.4.4 Field Collection Procedure

The study of each intersection requires simultaneous time measurements in order to calculate the average time in the system for each approach. Since people were available, the field set up was organized so three or four individuals could record all necessary measurements. The fourth individual was used only for the study of the busier approaches.

The individuals positioned themselves around each approach in a manner to best allow them to make their measurements, see Figure 4. Each individual had a specific job:

Individual 1: Conduct total intersection volume count, include volume count for each approach and record it every five minutes. Data recorded on Figure 5. Equipment: volume counter board.

Individual 2: Record vehicle color and arrival time (when the vehicle joins the queue), using cumulative time measurement, for each vehicle on the approach. Data recorded on Figure 6. Equipment: two stop watches.

Individual 3: Record service time (departure headway) by load type (L, N, or I), the time the vehicle leaves the intersection (cumulative time measurement), vehicle color

and type, and the turning movement made for each vehicle on the approach. Data recorded on Figure 7. Equipment: four stop watches.

Individual 4: This individual was only used at Intersection 1 for all approaches and at Intersection 2 for the southbound approach. He performed the same job as Individual 3. When needed, Individuals 3 and 4 alternated vehicles, recording data for every other vehicle on the approach.

Individuals 2, 3, and 4 synchronized their stop watches for the cumulative time measurements at the start of each study. This allowed for the calculation of average time in the system by subtracting the cumulative readings of Individual 2 (arrival time) from the cumulative readings of Individuals 3 and 4 (departure times). Vehicle colors were matched to ensure accuracy of the calculation.

2.4.5 Computation

All values that are required to validate the M/G/1 queuing model were measured in the field. Some values require manipulation of the recorded data. The values are obtained as follows:

1. Average Arrival Rate: obtained from the data recorded by Individual 2. Subtract consecutive cumulative arrival times to get the time elapsed between the arrival of each vehicle. Take the mathematical average of the time elapsed to get the average arrival rate.

2. Service Time (departure headway): time measurement recorded by Individuals 3 and 4. Computed separately for each type of intersection load (L, N, and I) and turn movement (L, T, R), yielding nine types of service time.

3. Variance of the Service Time: statistical calculation based on the service time measurements. Computed separately for each type of intersection load and turn movement.

4. Utilization Ratio: multiplication of the average arrival rate by the service time.

5. Average Time in the System: obtained from data recorded by Individuals 2, 3, and 4. Subtract departure time from arrival time (both are cumulative time measurements) to get the average time in the system.

Since there were three individuals making time measurements, it was necessary to make corrections for human perception differences. Each individual perceives a vehicle stop a little differently. In order to correct for this, duplicate field measurements were taken in the field.

I was Individual 3 for each approach study. In order to correct for Individual 4's measurements, we made several measurements for the same vehicles on the southbound approach of intersection 2. This duplication enabled the calculation of the mean difference between the cumulative measures of departure time and between the measures of service time. The mean differences for these two values

were added to all of Individual 4's measurements to determine "Nancy Equivalent" values for cumulative departure time and service time.

Calculation of "Nancy Equivalents" for the values measured by Individual 2 was conducted in a similar manner. When a vehicle arrives on the approach and there are no vehicles waiting in front of it, the cumulative arrival time should be equal to the cumulative departure time minus the service time, measurements made by Individual 3. The mean difference between the values recorded by Individuals 2 and 3 was then added to all the values measured by Individual 2 to determine the "Nancy Equivalent" average arrival time.

Before the data for Individuals 2 and 4 was adjusted, a Chi-Square test was conducted on the sample differences to check for normal distribution of the data. All differences are normal, so the appropriate mean adjustments to the data points could be accomplished. The "Nancy Equivalent" values were used for all subsequent calculations and analyses.

2.4.6 <u>Procedures for Data Analysis</u>

The volume count data is the first information examined. Volume splits are calculated separately for the four days of data collection at each intersection (one day for each approach) and for the total intersection volume. Percent turns are also computed for each intersection.

The next information examined is the departure headway, or service time, for each intersection. There are a total

of nine categories since the type of intersection load (L, N, and I) and the type of turn movements (left, through, and right) are considered separately.

Statistical significance tests are conducted between each category for each intersection. Significance tests are also run between the three intersections for each significantly different category. The two types of statistical tests used are the two-sided normal distribution test and the two-sided unpooled "t"-test. These tests determine whether or not there is a significant difference between the sample means of the two data sets.

The normal test is best for large samples. It is based on the Central Limit Theorem which states that for a sufficiently large sample size, n, the sampling distribution of $\overline{x}$ (the sample mean) is approximately normal, irrespective of the shape of the population distribution from which the sample is taken. The sampling distribution of the standardized variable

$$z = (\overline{x} - \mu)/(\sqrt{s \ / n}) \qquad (20)$$

is well-approximated by the z-curve (Devore and Peck, 1986). The minimum acceptable sample size for application of the Central Limit Theorem is n = 30. For smaller sample sizes, the unpooled "t"-test is more appropriate.

The justification for the procedures used with the normal test is invalid for small sample sizes because the Central Limit Theorem is not applicable. For small samples,

where n is less than thirty, the unpooled "t"-test is generally used. The "t"-test does not require the population values. The distribution curve is more spread out than the z-curve which is used by the normal test, so a greater difference in sample means is required to show a statistically significant difference. The test statistic used is:

$$t = (\bar{x} - \mu)/(\sqrt{s / n}) \tag{21}$$

with n - 1 degrees of freedom.

For both the normal test and the "t"-test, the terms are defined as follows:

z = normal test statistic, compared to z-curve to determine statistical significance,
t = "t" test statistic, compared to t-curve with n - 1 degrees of freedom to determine statistical significance,
n = number of data points in the sample,
$\bar{x}$ = sample mean,
μ = hypothesized value for the population mean, and
s = sample variance.

Significance tests are conducted at a 95 percent confidence level. The confidence level establishes the critical value for the determination of a significant difference in means. If the absolute value of the test statistic is greater than the absolute value of the critical value for the given confidence level, then the difference in means is statistically significant. The critical value (z-value) for the normal test is 1.96 at a 95 percent confidence level. The critical value for the "t"-test varies with the degrees of freedom. For example, at a 95

percent confidence level and 10 degrees of freedom the critical value is 2.23, and at 25 degrees of freedom the critical value is 2.06. As the degrees of freedom increase, the "t" critical value approaches the z critical value of 1.96, since the sample size is increasing and approaching a more normal distribution.

The 95 percent confidence level is most commonly used by investigators. This level establishes a difference without too much fear of error. A lower confidence level increases the chance of falsely showing a difference in means when no difference exists, and a higher level increases the chance of falsely showing no difference in means when a difference does exist.

The tests for statistical significance are set up as follows:

Ho: μ , the difference in sample means, is zero (there is no significant difference in the sample means)

Ha: μ is not zero (there is a significant difference in sample means)

Test Statistic:

$$z \text{ (or } t) = \frac{(\bar{x}_1 - \bar{x}_2)}{\sqrt{\dfrac{s_1}{n_1} + \dfrac{s_2}{n_2}}} \tag{22}$$

where

z = test statistic used for the normal test

t = test statistic used for the unpooled "t"-test
$\bar{x}_1$ = sample mean for data set 1
$\bar{x}_2$ = sample mean for data set 2
s_1 = sample variance for data set 1
s_2 = sample variance for data set 2
n_1 = sample size for data set 1
n_2 = sample size for data set 2

Rejection Region:

Reject the null hypothesis (Ho) if the test statistic is greater than the z (or t) critical value or if it is less than the -z (or -t) critical value. This is a two-tailed test. Values above or below the critical value are rejected with a 95% confidence that the true population means are different.

Critical Value:

z critical = 1.96 or -1.96 at a 95% confidence level

t critical is selected at the 95% confidence level for n - 1 degrees of freedom, where n is the sample size of the smaller sample in the test.

2.4.7 Comparison of Basic and Practical Capacities

If the measured service times are significantly different from those measured by Herbert for similar conditions, it is logical to recompute basic and practical capacities for four-way stop intersections. Methods similar to those described in the Herbert study are used to do this.

Basic capacity is the maximum number of passenger cars an intersection can process efficiently under ideal conditions. Practical capacity is approximately 80 percent of the basic capacity for a given intersection (HCM, 1985).

A linear equation can be calculated from the service time measurements, where service time is a function of traffic volume. Herbert derived the equation H = 10.15-5*S, where S is the ratio of the traffic volume on the major street to the total intersection volume, and H is the average service time (departure headway) for through

vehicles with loaded conditions (Herbert, 1963). This equation says that as volume split increases, service time decreases.

The data collected in this study enables the calculation of a similar linear equation. The value of average service time for through vehicles with loaded conditions is obtained for each intersection. Each intersection has a specific volume split, so three points on the line are established. A fourth point is obtained by using the total average service time for vehicles with non-loaded conditions. This is a reasonable approximation for a 100/0 volume split. Linear regression is used to solve the equation for the line established by these four points. The coefficient of determination (R squared) is examined to evaluate how well the data is approximated by a linear equation. Values of R squared that approach 1.0 say that the regression equation is a good representation of the data.

Basic intersection capacity can be calculated for various traffic volume splits with the following equation (Herbert, 1963):

$$\begin{array}{c}\text{Total}\\ \text{Intersection}\\ \text{Basic Capacity}\end{array} = \begin{array}{c}\text{volume on}\\ \text{loaded street}\end{array} + \begin{array}{c}\text{volume on}\\ \text{other street}\end{array}$$

$$= \frac{(3600)}{(10.15-5S)} * 2 + \frac{(3600)}{(10.15-5S)} * 2 * \frac{(1 - S)}{S}$$

$$= \frac{(7200)}{(10.15-5S)*S} \qquad (23)$$

If new values for the slope and y-intercept of the linear equation are discovered, these values are subsituted into this equation to derive new values of basic capacity. It is important to understand that these capacity calculations assume ideal conditions with simultaneous movement on opposite approaches. Herbert says,

> "The ... volumes are extremely high, and could only be attained under the most ideal conditions of roadway and traffic. The delays experienced by the waiting vehicles would be intolerably great. Under the best of prevailing conditions, and taking into account the effect of inept drivers and a variety of other factors, these theoretical capacities are impossible for most, if not all, four-way stop intersections. (Herbert, 1963)

2.4.8 Testing of the Pollaczek-Khintchine Formula

The Pollaczek-Khintchine Formula is used within the M/G/1 queuing model. This formula is (Richardson, 1987)

$$L = [2P - P^2 + \lambda^2 V(S)] / [2(1 - P)] \tag{8}$$

where

- L = average number in the system (average number on the approach, including the vehicle at the stop line),
- λ = average arrival rate,
- s = average service time,
- $V(S)$ = variance of service time, and
- P = utilization ratio, which is the arrival rate multiplied by the service time.

Little's equation is then used to determine the average time in the system

$$Ws = L / \lambda \tag{9}$$

where Ws is the average time in the system.

The Pollaczek-Khintchine formula is tested by substituting field data into the variables in the equation

and comparing the predicted average time in the system (Ws) with the average time in the system that was measured in the field. An unpooled "t"-test is performed to see if the difference in means between the predicted and measured values is significantly different from zero. If it is not significantly different, then the field data supports the accuracy of the formula. If it is significantly different, then the data shows that the formula does not accurately predict the average time in the system, at a 95 percent level of confidence.

2.4.9 Testing of the M/G/1 Queuing Model

The M/G/1 queuing model uses the Pollaczek-Khintchine formula to predict the average time in the system on the approach of a four-way stop. The only input data it requires is the traffic volume on all four approaches over the study period, the length of the study period, and the number of lanes on each approach. The average arrival rate is calculated by dividing the approach volume by the length of the study period (in seconds); the other values needed in the formula are predicted using the second part of the queuing model.

The second part of the model predicts the average and the variance of the service time for the approach being studied. These predictions are substituted into the Pollaczek-Khintchine formula and used to obtain a prediction of the average time in the system on the approach.

The accuracy of these predictions is evaluated separately from the predictions made by the Pollaczek-Khintchine formula. If either set of predictions are significantly different from the field measurements, it will be possible to determine which part of the queuing model is inaccurate.

The predictions of the average service time and the variance of the service time (the second part of the queuing model) are based on probabilities of crossflow traffic, minimum allowable headway, t(m), and total intersection clearance time, T(c).

Richardson takes the values for the minimum allowable headway and the intersection clearance time from Herbert's 1963 study. The minimum allowable headway, t(m), is equal to 4.0 seconds. It was obtained from the value of average headway for vehicles on an approach with no crossflow traffic (load Type N). The clearance time for each approach is given as t(c) = 3.6 + 0.1*(number of crossflow lanes from both directions). For a simple four-way stop with one lane on each approach, t(c) = 3.8, since there are two crossflow lanes (one from each direction). The total clearance time, T(c), is the sum of the clearance times on each approach. T(c) = 15.2 for the same simple four-way stop.

Richardson derives some equations to predict service time. For a simple four-way stop, they are:

$$\begin{aligned} Pew &= 1 - (1 - Pe)*(1 - Pw) \\ &= 1 - (1 - \lambda e*Se)*(1 - \lambda w*Sw) \qquad (23) \end{aligned}$$

$$Pns = 1 - (1 - Pn)*(1 - Ps)$$
$$= 1 - (1 - \lambda n*Sn)*(1 - \lambda s*Ss) \quad (24)$$

$$Sn = t(m)*(1 - Pew) + T(c)*(Pew) \quad (25)$$

$$Ss = t(m)*(1 - Pew) + T(c)*(Pew) \quad (26)$$

$$Se = t(m)*(1 - Pns) + T(c)*(Pns) \quad (27)$$

$$Sw = t(m)*(1 - Pns) + T(c)*(Pns) \quad (28)$$

where

- Pew = utilization ratio on the east-west approaches
- Pns = utilization ratio on the north-south approaches
- Pe = utilization ratio on the east approach (service time on the east approach times the approach volume divided by the total volume) [Pw, Pn, and Ps are similarly defined]
- Sn = service time on northbound approach [Ss, Se, and Sw are similarly defined]
- t(m) = 4.0 seconds
- T(c) = 15.2 seconds

A detailed explanation of the derivation of these equations is contained in the Background on the Richardson M/G/1 Queuing Model section.

These equations can be solved using Richardson's assumption of equal service time on opposite approaches. He says that the average service time on the northbound approach and the southbound approach are functions of the flows and service rates on both the eastbound and westbound approaches; the service time on the northbound approach will be the same as the service time on the southbound approach since vehicles heading north and south have to give way to exactly the same eastbound and westbound traffic (Richardson, 1987). The same holds true for the service time on the eastbound and westbound approaches.

Using this information, the equation for the utilization ratio becomes a function of one value for service time. This equation is then substituted into the service time equation. The service time equation can now be solved quadratically to determine the predicted service time on the approach.

Once the average service time is known, the variance can be calculated as follows (Richardson, 1987):

$$V(S) = \frac{t(m)^2 * [T(c) - S)}{T(c) - t(m)} + \frac{T(c)^2 * [S - t(m)]}{T(c) - t(m)} - S^2 \qquad (17)$$

The values of the average and the variance of the service times are now substituted into the Pollaczek-Khintchine formula and Little's equation to obtain a prediction of the average time in the system on each approach for each intersection.

Predictions of average time in the system on each approach are calculated and then compared to actual values measured in the field. An unpooled "t"-test is used to see if there is a statistically significant difference in the means and thereby determine the accuracy of the model.

2.4.10 Testing the M/G/1 Queuing Model With Revised Service Time

Since a different value for average service time (departure headway) for a Type N intersection load is expected, we wanted to determine how such a difference affects the M/G/1 queuing model.

The Pollaczek-Khintchine formula is not affected. Any difference becomes important in the second half of the model with the use of t(m), the minimum allowable headway. If the average service time is different from 4.0, the measured value for t(m) is substituted back into the equations that predict the average and the variance of the service time. These results are used to predict the average time in the system on each approach for each intersection.

The accuracy of the predictions made by the queuing model and the revised model are compared to the field data and to each other. The predicted values for each approach are subtracted from the field values for those same approaches. This is done for the Pollaczek-Khintchine formula, the queuing model, and the revised queuing model. The accuracy of each type of prediction is evaluated with an unpooled "t"-test at a 95 percent level of confidence. The M/G/1 queuing model and the revised queuing model are compared to each other by checking their respective average difference from the field values. Trends in prediction accuracy are examined by looking at turning movements, traffic volumes, and other factors that affect the model assumptions and therefore the ability of the model to make accurate predictions.

2.5 Presentation and Discussion of the Results

2.5.1 Intersection Volume Counts

The volume counts for each approach of each intersection are given in Table 8. Since results are collected over a four-day period for each intersection (one day for each approach), it is possible to analyze the validity of the assumption that traffic patterns are the same over the days of data collection.

The volume counts for each intersection support this assumption. Intersection 2 has an average north-south/east-west volume split of 0.56 / 0.44 with a 3.4 percentage points difference over the four days of data collection. Intersection 3 has an average north-south/east-west split of 0.38 / 0.62 with 5.0 percentage points difference over the four days of data collection. Intersection 1 has an average north-south/east/west split of 0.54 / 0.46. It has the most variation, with 11.8 percentage points difference. This variability could be due to construction a few miles west of the intersection. The entrance from another primary road to Park Road was blocked, so the eastbound traffic at the intersection of Park and Worthington-Galena may have been less than normal. The construction existed when data was collected for the northbound and for the eastbound approaches. The construction has a minimal effect on the total intersection volume and it should also have a minimal effect (if any) on the recorded service times.

2.5.2 Significant Differences in Service Times

Service times, or departure headways, were measured at each intersection. Average values are calculated for each type of intersection load (L, N, and I) and for each type of turn movement (left, through, and right) at each intersection.

Both normal and "t"-tests for statistically significant differences in means are run on the data for each intersection at a 95 percent confidence level. Significance tests are done for each type of turn movement within each type of intersection load and for each type of load within each type of turn movement. The results are shown by the lines diagrams in Figure 9. Interpret the lines diagrams as follows: when a line is drawn beneath two categories, there is no significant difference in means.

These results are qualitatively very similar to the results in the Herbert study, as expected. They show that left turns have no effect on service time but right turns usually do have an effect on service time. For intersection load type L, service times for all types of turning movements are significantly different from each other. And finally, intersection volume split has no obvious effect on service time. It must be noted that significant differences can also be influenced by sight distance, intersection geometrics, and other characteristics specific to each intersection.

Quantitatively, the results are quite different from those in the Herbert study, also as expected. The measured average service times for each intersection are less than the service times reported in the Herbert study. The measured results for each intersection and for the intersections combined are given in Table 9. They are listed by load type, by turn movement, and by total average service time for each intersection (or combination).

Since Intersection 3 has a volume split and percent turning movements that are very similar to Intersection B in the Herbert study, a direct comparison is made between these two intersections. The measured values and the statistical significance test results are shown in Table 10.

Both the normal test and the unpooled "t"-test show that the mean values for average service times are significantly different for all three types of intersection load. It is therefore possible to assume that the average service time for the type N intersection load is no longer 4.0 seconds, as reported in the Herbert study. Current field measurements show a current value of 2.58 seconds for the average service time for an intersection with a type N load (where there are no vehicles on the cross- street when a vehicle arrives at the stop line). This value becomes important in the evaluation of the Richardson M/G/1 queuing model.

2.5.3 Revision of Basic and Practical Capacities

The service time measurements are also used to make basic and practical intersection capacity calculations. The regression equation of the data yields the following equation: $H = 7.79 - 5.13*S$, with an R squared value of 0.9997 (see Table 11). This means the regression equation does a very good job of approximating the data. However, the high value of R squared could be due in part to the small sample size. The equation maintains the relationship where an increase in volume on the major street results in a decrease in service time.

Basic capacity and practical capacity calculations for the field data are different from those reported in the 1985 Highway Capacity Manual. A comparison of the two sets of values are shown in Table 12. The capacity values calculated from the field data are extremely high. The process to determine capacity assumes simultaneous movement on opposite approaches. It also assumes ideal conditions, and I agree with Herbert's opinion that such values could never be attained in the field.

Intersection 1 has the two busiest approaches (northbound and westbound) of all that were studied. The northbound approach experienced loaded conditions (Type L) about 41 percent of the time and westbound experienced loaded conditions about 57 percent of the time. The northbound approach had volume count of 219 vehicles in 30

minutes. The westbound approach had a volume count of 294 vehicles in one hour. By multiplying vehicles per hour by 4 (there are four approaches) and dividing by the percent loaded, it is possible to come up with a rough estimate of field capacity. This estimate is subject to error since multiplying by four assumes all four approaches will behave in the same way.

The westbound approach calculations give a capacity of 2063 vph. This value is less than the predicted basic capacity, but it is greater than Herbert's basic capacity. The northbound approach calculations give a capacity of 4273 vph. This is an extremely high number and it would not occur for actual conditions. The northbound approach has a high percent of right turning vehicles, over 50 percent. Field observation showed that right turning vehicles would enter the intersection at the same time as a through or left vehicle. This would effectively double the intersection capacity, almost making it a two-lane approach. Dividing the capacity by four gives 2136 vph. This value is more consistent with the value for the westbound approach.

These capacity approximations are significantly higher than Herbert's basic capacity calculations to warrant further investigation.

2.5.4 Validation of the Pollaczek-Khintchine Formula

Predictions for average time in the system are made with the Pollaczek-Khintchine formula using field

measurements as input variables. These predictions for each approach of every intersection are compared to the measured field values of average time in the system on the same approaches (see Table 13). A paired "t"-test shows that there is no significant difference between the predictions made by the formula and the field values, at a 95 percent level of confidence. In other words, the field data supports the accuracy of the Pollaczek-Khintchine formula.

2.5.5 Validation of the M/G/1 Queuing Model

The M/G/1 queuing model makes predictions of average time in the system for each approach using only the volume count and the number of lanes in the approach. Values for the average and variance of the service times are predicted as described in the Methodology Section. The predictions of average time in the system are compared to the measured field values in Table 13. A paired "t"-test shows that there is no significant difference between the predicted values and the measured values at a 95 percent confidence level. The field measurements support the accuracy of the model predictions.

2.5.6 The M/G/1 Model With Revised Minimum Allowable Headway

Although the statistical tests show that the M/G/1 queuing model is fairly accurate, the minimum allowable headway used by the model (4.0 seconds) is significantly

greater than the minimum allowable headway currently existing in the field. For this reason, it is beneficial to determine the effect of the lower headway (2.58 seconds) on the accuracy of the queuing model.

The value of 2.58 seconds is used in place of 4.0 seconds in the calculations for the predictions of average time in the system. These predictions are compared to the field measurements and to the predictions made by the unrevised model. The paired "t"-test shows that the accuracy of the model is supported by the field data; there is no significant difference between the two values.

It is interesting to note that there is a significant difference between the predictions made by the original M/G/1 queuing model and by the revised model. The average value of the difference between the prediction and the field measurement is significantly greater for the unrevised model. This indicates that although both models are shown to be accurate, the revised model with the current value for minimum allowable headway is more accurate than the unrevised model.

2.5.7 Interpretation of Model Accuracy

Both the original M/G/1 queuing model and the revised model are fairly accurate. However, examination of the different predictions as compared to the field data yields the following observations. Both models, as well as the Pollaczek-Khintchine formula, underestimate the average time

in the system for the westbound approach of Intersection 1. The two models overestimate the average time in the system for the east and westbound approaches of Intersection 3 by a fairly large amount. And, the original queuing model overestimates the average time in the system for the north and southbound approaches of Intersection 2 while the revised model does not. Traffic volume and turn movement data provide possible explanations for these especially poor predictions (Table 14).

The first case involves the underestimation of the average time in the system for the westbound approach of Intersection 1. Since both models and the Pollaczek-Khintchine formula underestimate the field data by approximately the same amount, it appears that the error in this case is in the Pollaczek-Khintchine formula. This particular approach experiences a very high percent of left turns (45%). Also, most vehicles experience load type L as they progress through this approach. Service times for left turns with load type L are significantly longer than for through or right turn movements. This would account for the underestimation by the formula and the models. Another possibility is that this approach is busier than some of the others. Richardson cautions that the model does not predict as well as volume approaches capacity, because the assumption of random arrivals becomes suspect. The poor prediction could be a result of the approach conditions not

meeting the assumptions.

The second case involves the overestimation of average time in the system for the east and westbound approaches of Intersection 3. Since the formula predicts this value well, it appears that the error is in the model's ability to predict the average and the variance of the service time. These two predictions are based on the cross-flow (north-south) traffic. One assumption made by Richardson in the development of the model is that the approach volumes are approximately equal on opposite approaches. Here the northbound traffic volume is about five times the volume on the southbound approach (Table 7). Evidently, extreme imbalances in traffic volume cause the model predictions to be inaccurate.

Finally, average time in the system is overestimated on the north and southbound approaches of Intersection 2 by the original M/G/1 queuing model only. Apparently, the corrected minimum allowable headway had a greater effect on the accuracy of these two approaches than on any others. The original model predicted very high service times for these two approaches as compared to its service time predictions for the other approaches. Perhaps it is in some way related to the turning movements; more likely it is a result of the revised minimum allowable headway giving more realistic service time predictions, here almost half the values predicted by the unrevised model.

The data shows that the M/G/1 queuing model is fairly accurate. However, it also shows that when the basic assumption of equal volume on opposite approaches is not met, the predictions are not as accurate. The model is also unable to account for the effect of turning movements, which can have a substantial effect on the average time in the system (Table 14). Fortunately, at the time Richardson published the information on his queuing model, he was developing another model that incorporates turn movement data. This leaves only the problem of ensuring the traffic volumes are reasonably close for opposite approaches at the intersection being studied. Realistically, if the approach volumes are not close, a four-way stop is probably not warranted.

2.6 Conclusions and Recommendations

The data collected and analyzed in this study yields the following two items of interest. First, the average service times, or departure headways for today's drivers are significantly different than the values for drivers in 1963. Second, the predictions of average time in the system made by the Richardson M/G/1 queuing model are not significantly different from the measured lengths of time a vehicle waits in a queue on an approach at a four-way stop intersection.

2.6.1 Service Time Conclusions

The average service times are almost half the times reported by Herbert in 1963. For the three intersections in this study, the average service time for an intersection with load type L is 4.76 seconds, load type N is 2.58 seconds, and load type I is 2.75 seconds. The average service time for left turning vehicles is 3.78 seconds, through vehicles is 3.51 seconds, and right turning vehicles is 3.13 seconds. These values are specific to this area and may not be accurate for vehicles in other parts of the country.

Following the same methods described in the Herbert study, new values are calculated for basic and practical intersection capacity. These values are substantially greater than the capacities defined in the Herbert study. This indicates that today's drivers accelerate faster or the cars perform more quickly, or a combination of both. In any case, this information suggests that a greater capacity of vehicles can be handled at a four-way stop than the information in the 1985 Highway Capacity Manual dictates.

2.6.2 M/G/1 Queuing Model Conclusions

The data collected in this study supports the accuracy of Richardson's M/G/1 queuing model predictions of average time in the system on an approach at a four-way stop. Although the predictions from the existing model are accurate, the predictions from the revised model with a

minimum allowable headway of 2.58 seconds is more accurate. (The original model uses a minimum allowable headway of 4.0 seconds.)

The implications of the support by field data are that the model is reliable and it can be used to predict delay and determine level of service at the four-way stop intersection. To determine level of service, simply add values for acceleration and deceleration time to the predicted average time in the system. This value can be compared to the recommended delays for each level of service which are given in the 1985 Highway Capacity Manual. For example, at level of service C a delay of 30 seconds is given. If your intersection has a speed limit of 35 mph, then deceleration delay is 4.5 seconds (at a rate of 27.5 ft/sec) and acceleration delay is 7.0 seconds (at a rate of 5 mph). This leaves approximately 19.5 seconds for average time in the system.

It is important to note that this model is not as reliable as traffic flows approach capacity, since the assumption of random vehicle arrivals becomes questionable. Richardson cautions,

> "As with all delay models based on queuing theory principles, care should be taken when interpreting the delays predicted when the flow approaches the capacity. Because of the extreme sensitivity of delay to changes in flow in this region, the delay values predicted should be used only in a diagnostic fashion and should not be interpreted literally." (Richardson, 1987)

2.6.3 Recommendations

This study shows that the Richardson M/G/1 queuing model is a valid method of predicting delay at four-way stop intersections. The model has been validated by a simulation model in the past, and now field measurements also support its predictions. It is preferable over the Table 10-5 in the 1985 Highway capacity manual since it can make predictions for more specific traffic volume splits. The model is also able to reproduce the results obtained from a validated simulation model with the added ability of predicting levels of performance over a wider range of operating conditions, without the need for detailed simulations.

A similar model that incorporates information on turn movements should be developed, but since field information is not always available, the M/G/1 queuing model should be used as an approved method of predicting delay.

Although the data collected is limited in scope and in regional area, the different practical capacity values from those given in the 1985 Highway Capacity Manual should be noted. More research should be done in this area to update the information in the 1985 Highway Capacity Manual.

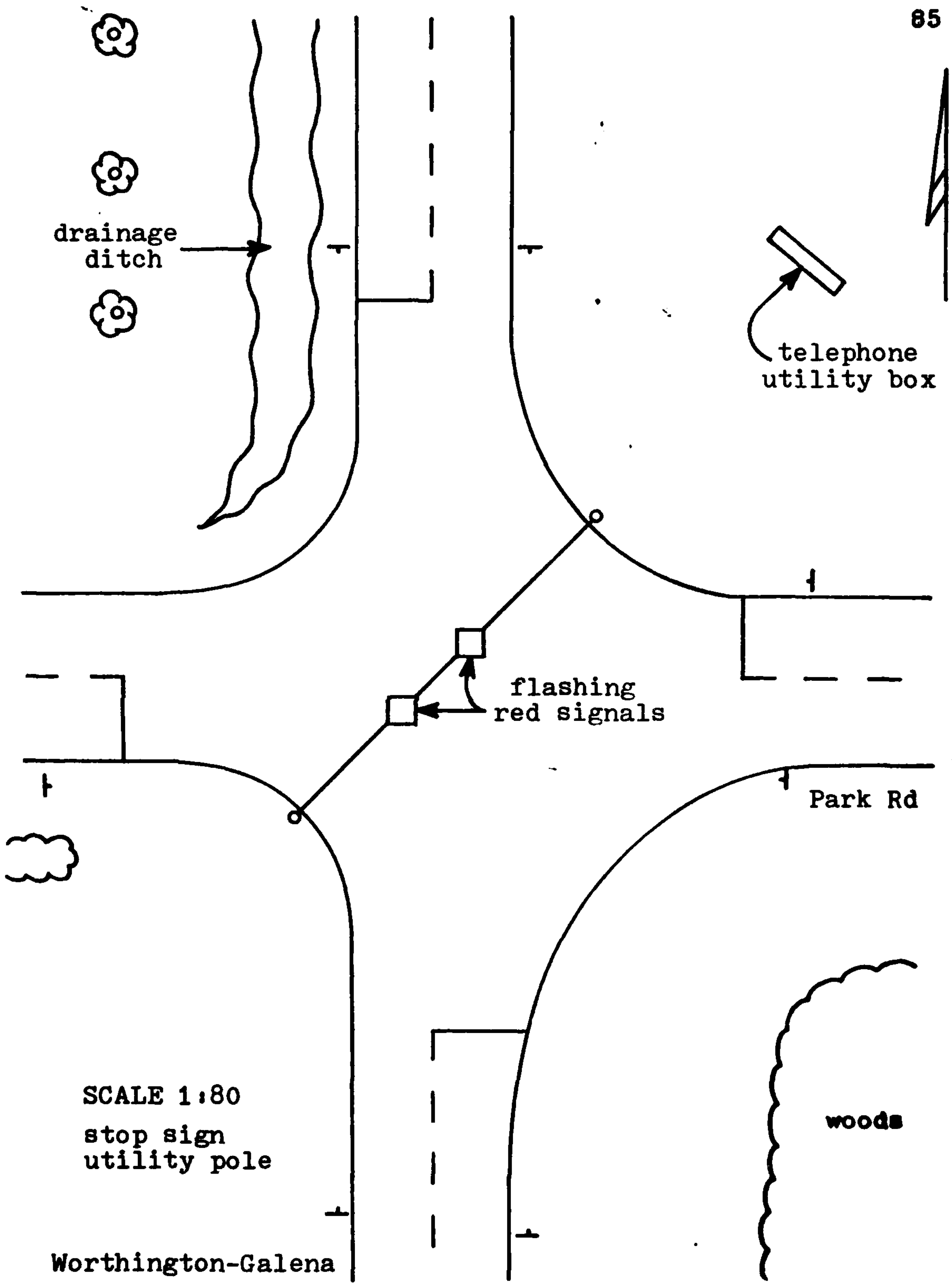

Figure 1: Intersection 1, Park Rd and Worthington-Galena Rd Columbus, Ohio

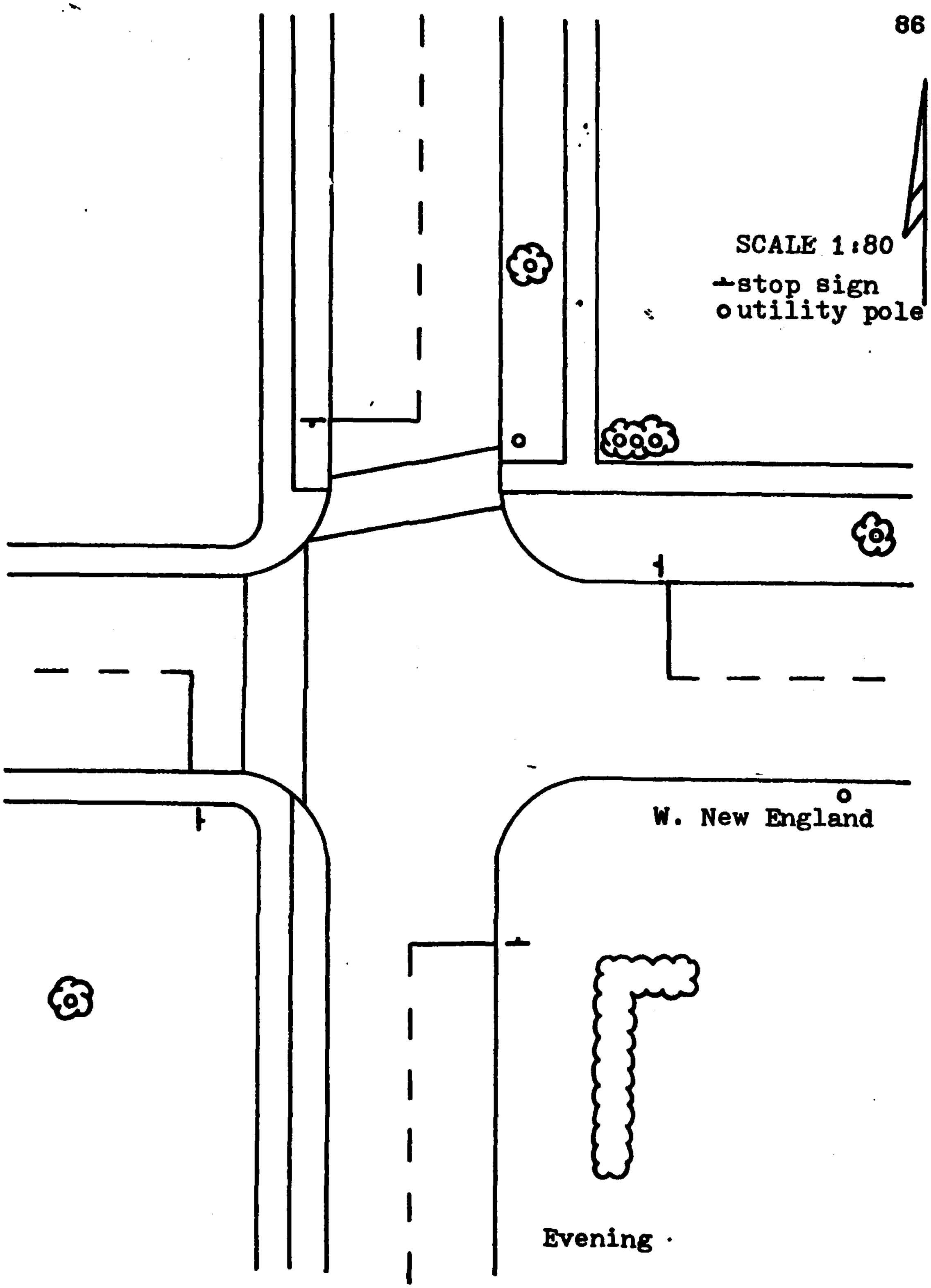

Figure 2: Intersection 2, W. New England Ave and Evening St
Worhtington, Ohio

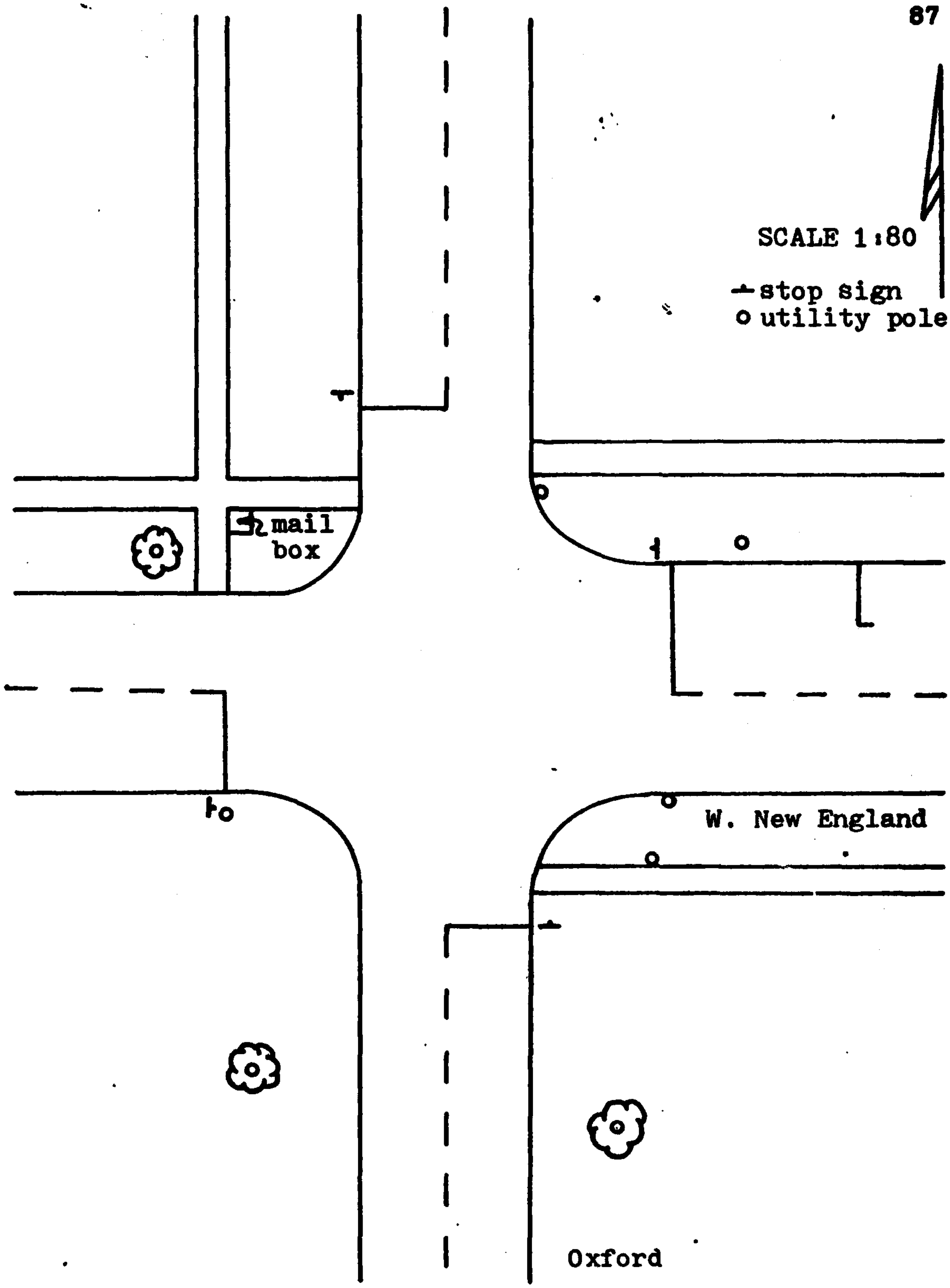

Figure 3: Intersection 3, W. New England Ave and Oxford St Worthington, Ohio

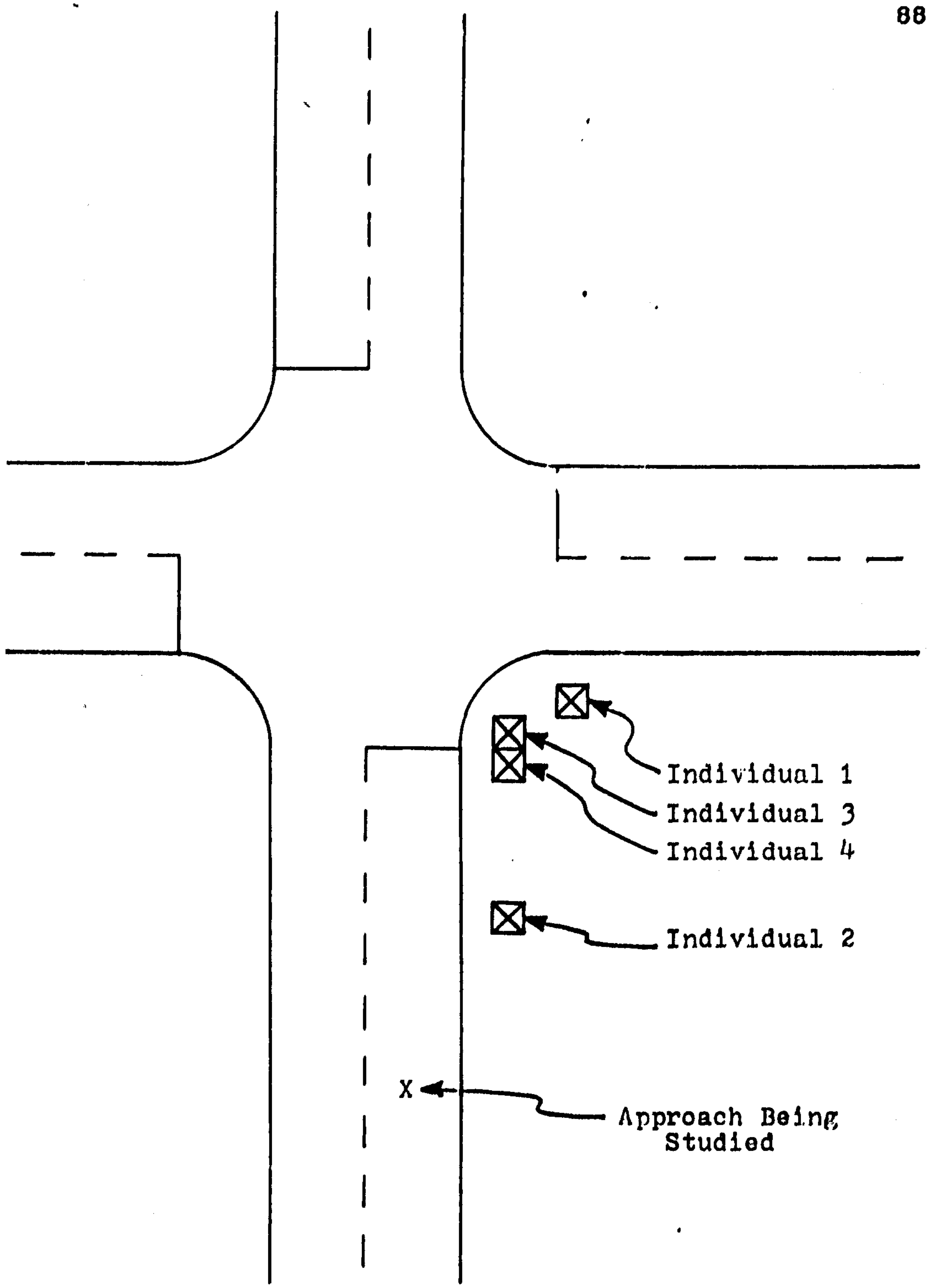

Figure 4: Individual Locations for Data Collection

TRAFFIC VOLUME AND
AVERAGE NUMBER OF VEHICLES ON THE APPROACH

LOCATION: ____________________

DATE: ____________ TIME: ____________ PG 1 OF ___

HOUR	WB VOL	NB VOL	EB VOL	SB VOL
____				
__00				
05				
10				
15				
20				
25				
__30				
35				
40				
45				
50				
55				
__00				
05				
10				
15				
20				
25				
__30				
35				
40				

Figure 5: Data Collection Sheet for Individual 1

AVERAGE ARRIVAL RATE

LOCATION: ______________________________

DATE: ______________ TIME: __________ PAGE 1 OF ____

VEH COLOR	CUMULATIVE TIME	REDUCED TIME	VEH COLOR	CUMULATIVE TIME	REDUCED TIME
1			23		
2			24		
3			25		
4			26		
5			27		
6			28		
7			29		
8			30		
9			31		
10			32		
11			33		
12			34		
13			35		
14			36		
15			37		
16			38		
17			39		
18			40		
19			41		
20			42		
21			43		

Figure 6: Data Collection Sheet for Individual 2

AVERAGE DEPARTURE HEADWAY/SERVICE TIME

LOCATION: ______________________________ APPROACH: __________

DATE: __________________ TIME: ____________ PAGE 1 OF ___

N = no vehicles approaching on cross street (within 50 feet)
L = loaded, one vehicle accelerating from cross street approach within the recorded headway
I = interference, vehicle on cross street within 50 feet of stop line (only causes hesitation for vehicle on the approach)

HEADWAY	CUM TIME	LOAD	COLOR	TURN
1 ____ . ____	____ . ____ . ____	____	____	____
2 ____ . ____	____ . ____ . ____	____	____	____
3 ____ . ____	____ . ____ . ____	____	____	____
4 ____ . ____	____ . ____ . ____	____	____	____
5 ____ . ____	____ . ____ . ____	____	____	____
6 ____ . ____	____ . ____ . ____	____	____	____
7 ____ . ____	____ . ____ . ____	____	____	____
8 ____ . ____	____ . ____ . ____	____	____	____
9 ____ . ____	____ . ____ . ____	____	____	____
10 ____ . ____	____ . ____ . ____	____	____	____
11 ____ . ____	____ . ____ . ____	____	____	____
12 ____ . ____	____ . ____ . ____	____	____	____
13 ____ . ____	____ . ____ . ____	____	____	____
14 ____ . ____	____ . ____ . ____	____	____	____
15 ____ . ____	____ . ____ . ____	____	____	____
16 ____ . ____	____ . ____ . ____	____	____	____
17 ____ . ____	____ . ____ . ____	____	____	____
18 ____ . ____	____ . ____ . ____	____	____	____

Figure 7: Data Collection Sheet for Individuals 3 and 4

INTERSECTION 1: PARK AND WORTHINGTON-GALENA

Load	Turn Movement	
L	L T R	right turn sig diff from left and through
N	L T R	right turn sig diff from left and through
I	L T R	no sig diff betw type I turn movements

Turn Mvmt	Load Type	
L	L N I	L sig diff from N and I for left turns
T	L N I	L sig diff from N and I for through mvmts
R	L N I	L sig diff from N and I for right turns

INTERSECTION 2: WEST NEW ENGLAND AND EVENING

Load	Turn Movement	
L	L T R	no sig diffs between type L turn mvmts
N	L T R	no sig diffs between type N turn mvmts
I	L T R	right sig diff from left and through

Turn Mvmt	Load Type	
L	L N I	L sig diff from N and I for left turns
T	L N I	L sig diff from N and I for through mvmts
R	L N I	L sig diff from N and I for right turns

INTERSECTION 3: WEST NEW ENGLAND AND OXFORD

Load	Turn Movement	
L	L T R	sig diff betw type L left and right turns
N	L T R	no sig diff between type N turn mvmts
I	L T R	no sig diff between type I turn mvmts

Turn Mvmt	Load Type	
L	L N I	L sig diff from N and I for left turns
T	L N I	L sig diff from N and I for through mvmts
R	L N I	L sig diff from N and I for right turns

Figure 8: Statistical Significance Between Load Types and Turn Movements for Each Intersection

Table 2: Headways of Passenger Cars Entering A Four-Way Stop Intersection (Herbert, 1963)

INTERSECTION	MOVEMENT	TYPE OF HEADWAY	MEAN (sec)	STD DEV (sec)
A	Through	N	3.81	1.61
	Through	I	4.73	1.86
	Through	L	7.85	2.09
	Left	L	7.40	2.22
	Right	L	5.40	2.05
B	Through	L	6.90	1.60
	Through	L	7.04	1.18
	Through	N	4.18	1.36
	Through	I	4.28	1.62
	Through	L	6.96	1.51
	Left	L	7.57	2.12
	Right	L	6.38	2.06
A & B	Through	L	7.32	1.89
	Left	L	7.45	2.19
C	Through	L	8.08	1.03

Table 3: Capacities for Four-Way Stops With Various Traffic Volume Splits (Herbert, 1963)

VOLUME SPLIT	BASIC CAPACITY (vph)	PRACTICAL CAPACITY (vph)
50/50	1900	1370
55/45	1800	1300
60/40	1700	1230
65/35	1600	1150
70/30	1550	1100

Table 4: Capacity of a Four-Way Stop Intersection (Richardson, 1987)

DEMAND SPLIT	1985 HCM CAPACITY (vph)	DELAY MODEL PREDICTED CAPACITY (vph)
50/50	1900	1900
55/45	1800	1760
60/40	1700	1650
65/35	1600	1600
70/30	1500	1560
80/20		1520
90/10		1570
100/0		1800

Table 5: Total Intersection Delays Predicted By Delay Model for LOS C Flow Combinations From the 1985 HCM (Richardson, 1987)

	TWO-BY-TWO LANES		FOUR-BY-FOUR LANES	
DEMAND SPLIT	TOTAL FLOW	DELAYS	TOTAL FLOW	DELAYS
50/50	1200	24.4/24.4	2200	26.9/26.9
55/45	1140	24.3/23.1	2070	24.2/22.4
60/40	1080	23.9/22.2	1970	27.4/21.6
65/35	1010	22.7/20.0	1880	27.1/21.1
70/30	960	21.9/19.8	1820	26.9/20.8

Table 6: Minimum Sample Size Calculations

VARIABLE	TYPE L	TYPE N	TYPE I
n	14	29	11
x	4.009	2.917	3.161
s	1.165	1.350	1.093
0.5 accuracy			
B	0.50	0.50	0.50
N	21	28	19
0.25 accuracy			
B	0.25	0.25	0.25
N	84	112	74
0.1 accuracy			
B	0.10	0.10	0.10
N	521	701	459

Table 7: Intersection Data Collection Dates and Times

	NORTH BOUND APPROACH	SOUTH BOUND APPROACH	EAST BOUND APPROACH	WEST BOUND APPROACH
INT 1				
DATE	10/26/88	9/6/88	10/27/88	9/7/88
START	1200	1200	1200	1210
STOP	1230	1300	1230	1300
INT 2				
DATE	9/14/88	9/6/88	9/7/88	9/20/88
START	1340	1340	1330	1315
STOP	1500	1500	1500	1500
INT 3				
DATE	9/20/88	9/14/88	9/6/88	9/7/88
START	1520	1520	1525	1525
STOP	1700	1700	1625	1700

Table 8: Intersection Volume Counts and Turn Data

INTERSECTION 1: PARK RD AND WORTHINGTON-GALENA RD

APPROACH	NB VOL	SB VOL	EB VOL	WB VOL	% N/S SPLIT	% E/W SPLIT
NB	219	94	65	133	61.25	38.75
SB	222	178	137	228	52.29	47.71
EB	217	103	82	154	57.55	42.45
WB	277	185	179	294	49.41	50.59
Total	935	560	463	809	54.03	45.97

TOTAL INTERSECTION TURN MOVEMENTS		
	Left	26.53 %
	Through	48.98 %
	Right	24.49 %

INTERSECTION 2: WEST NEW ENGLAND AVE AND EVENING ST

APPROACH	NB VOL	SB VOL	EB VOL	WB VOL	% N/S SPLIT	% E/W SPLIT
NB	203	207	136	154	58.57	55.21
SB	178	195	115	175	56.26	43.74
EB	172	228	144	173	55.79	44.21
WB	236	246	160	231	55.21	44.79
Total	789	876	555	733	56.38	43.62

TOTAL INTERSECTION TURN MOVEMENTS		
	Left	19.06 %
	Through	53.90 %
	Right	27.04 %

INTERSECTION 3: WEST NEW ENGLAND AVE AND OXFORD ST

APPROACH	NB VOL	SB VOL	EB VOL	WB VOL	% N/S SPLIT	% E/W SPLIT
NB	253	841	172	850	35.67	37.80
SB	221	39	240	183	38.07	61.93
EB	137	30	136	108	40.63	59.37
WB	230	61	245	223	38.34	61.66
Total	841	172	850	817	37.80	62.20

TOTAL INTERSECTION TURN MOVEMENTS		
	Left	19.70 %
	Through	62.68 %
	Right	17.68 %

Table 9: Average Service Times By Intersection Load and Turn Type

INTERSECTION 1: PARK RD AND WORTHINGTON-GALENA RD

	HEADWAY TYPE			TURN MOVEMENT			
	L	N	I	L	T	R	TOTAL
MEAN	4.82	2.57	2.77	4.28	4.24	3.10	3.92
VAR	3.96	1.18	1.57	4.35	4.13	2.45	3.97
COUNT	378	81	203	148	323	193	664

INTERSECTION 2: WEST NEW ENGLAND AVE AND EVENING ST

	HEADWAY TYPE			TURN MOVEMENT			
	L	N	I	L	T	R	TOTAL
MEAN	4.85	2.58	2.71	3.45	3.27	3.22	3.29
VAR	3.96	1.01	0.89	3.26	2.96	2.46	2.89
COUNT	164	244	143	105	297	149	551

INTERSECTION 3: WEST NEW ENGLAND AVE AND OXFORD ST

	HEADWAY TYPE			TURN MOVEMENT			
	L	N	I	L	T	R	TOTAL
MEAN	4.52	2.58	2.74	3.45	3.06	3.06	3.14
VAR	2.26	0.87	1.25	2.79	1.88	1.55	2.03
COUNT	155	267	172	117	372	105	594

COMBINED INTERSECTION DATA

	HEADWAY TYPE			TURN MOVEMENT			
	L	N	I	L	T	R	TOTAL
MEAN	4.76	2.58	2.75	3.78	3.51	3.13	3.47
VAR	3.60	0.97	1.28	3.72	3.21	2.25	3.13
COUNT	697	592	518	370	992	447	1807

Table 10: Comparison of Service Times, 1963 vs 1986
(1963 values from Herbert, 1963)

INTERSECTION	TURN MVMT (L,T,R)	LOAD TYPE (L,N,I)	MEAN (sec)	VARIANCE (sec)	COUNT (n=)
INT B	Through	L	6.96	2.28	75
(1963)	Through	N	4.18	1.85	87
	Through	I	4.28	2.62	70
	Left	L	7.57	4.49	21
	Right	L	6.38	4.24	17
INT 3	Through	L	4.49	2.22	88
(1988)	Through	N	2.55	0.88	180
	Through	I	2.73	1.05	104
	Left	L	4.91	2.49	41
	Right	L	4.04	1.53	26

STATISTICAL TESTING

COMPARE	TEST STAT	Z CRITICAL	t CRITICAL	SIG DIFF?
Through L	10.49	1.96	2.00	Yes
Through N	10.09	1.96	1.99	Yes
Through I	7.00	1.96	2.00	Yes
Left L	5.08	1.96	2.09	Yes
Right L	4.22	1.96	2.12	Yes

Table 11: Determination of New Capacities, Regression Equation and Calculations

MAJ ST VOL/ TOT INT VOL	LOADED HEADWAY
0.5403	4.9520
0.5638	4.8208
0.6220	4.4855
1.0000	2.5807

REGRESSION OUTPUT

Constant	= 7.7918
Std Err of Y Est	= 0.0225
R Squared	= 0.9997
No. of Observations	= 4.0000
Degrees of Freedom	= 2.0000
X Coefficient	= -5.1306
Std Err of Coef.	= 0.0675

% VOL MAJ ST	CALC HEADWAY	INTERSECTION CAPACITY	BASIC CAPACITY	PRACTICAL CAPACITY
0.50	5.23	2755	2750	1980
0.55	4.97	2634	2630	1890
0.60	4.71	2546	2540	1830
0.65	4.46	2485	2480	1780
0.70	4.20	2449	2450	1760
0.75	3.94	2434	2430	1750
0.80	3.69	2441	2440	1760
0.85	3.43	2469	2470	1780
0.90	3.17	2520	2520	1810
0.95	2.92	2598	2600	1870
1.00	2.66	2706	2700	1940

Table 12: Capacity at Four-Way Stop Intersections

	1985 HCM		REVISED	
VOLUME SPLIT	BASIC CAPACITY	PRACTICAL CAPACITY	BASIC CAPACITY	PRACTICAL CAPACITY
50/50	1900	1370	2750	1980
55/45	1800	1300	2630	1890
60/40	1700	1230	2540	1830
65/35	1600	1150	2480	1780
70/30	1550	1100	2450	1760
75/25			2430	1750
80/20			2440	1760
85/15			2470	1780
90/10			2520	1810
95/05			2600	1870
100/0	1800		2700	1940

Table 13: Comparison of Results, Field Data vs Model Predictions of Average Time in the System (Ws)

PREDICTED TIME IN THE SYSTEM

INTERSECTION	FIELD RESULTS (sec)	P-K FORMULA (sec)	M/G/1 Q.MODEL (sec)	M/G/1 REVISED t(m)=2.58
PARK & W-G				
North	1.7124	1.1555	3.4135	2.9320
South	3.3067	1.6543	2.1954	1.4572
East	1.4880	1.3318	1.8712	1.0279
West	6.8780	1.5844	1.9741	1.1675
NE & EVENING				
North	1.6268	1.2608	7.4754	1.8172
South	1.3297	1.2392	7.3624	1.7819
East	0.9177	1.0595	1.6603	0.6995
West	1.4131	1.2266	1.6321	0.6550
NE & OXFORD				
North	1.2481	1.3174	1.4416	0.2928
South	0.9252	1.4419	1.5064	0.4152
East	1.1655	1.1568	6.6129	6.5274
West	1.3773	1.1051	5.8178	5.7118

DIFFERENCE BETWEEN PREDICTION AND FIELD MEASUREMENT

INTERSECTION	P-K FORMULA (sec)	M/G/1 Q.MODEL (sec)	M/G/1 REVISED t(m)=2.58
PARK & W-G			
North	-0.5569	1.7011	1.2196
South	-1.6524	-1.1113	-1.8495
East	-0.1562	0.3832	-0.4601
West	-5.2936	-4.9039	-5.7105
NE & EVENING			
North	-0.3660	5.8486	0.1904
South	-0.0905	6.0327	0.4522
East	0.1418	0.7426	-0.2182
West	-0.1865	0.2190	-0.7581
NE & OXFORD			
North	0.0693	0.1935	-0.9553
South	0.5167	0.5812	-0.5100
East	-0.0087	5.4474	5.3619
West	-0.2722	4.4405	4.3345
MEAN =	-0.6546	1.6312	0.0914
VARIANCE =	2.2073	9.7800	7.2669
n =	12.0000	12.0000	12.0000

Table 14: Percent Turn Movements on Each Approach

INTERSECTION	TOTAL FLOW	PERCENT LEFT TURNS	PERCENT THROUGH TURNS	PERCENT RIGHT TURNS
PARK & W-G				
North	118	12.71	36.44	50.85
South	131	21.37	57.25	21.37
East	79	6.33	62.03	31.65
West	230	45.22	42.61	12.17
NE & EVENING				
North	158	43.67	52.53	3.80
South	161	28.57	68.94	2.48
East	111	3.60	46.85	49.55
West	187	3.74	40.64	55.61
NE & OXFORD				
North	212	42.92	15.57	41.51
South	38	57.89	23.68	18.42
East	100	3.00	94.00	3.00
West	118	2.68	50.53	15.25

CHAPTER III

THE USE OF FOUR-WAY STOPS IN THE FIELD

3.1 Interviews With Personnel in the Field

The use of four-way stops as a method of traffic control is historically controversial. I have noticed communities in and around Columbus that use four-way stops frequently while others do not use them at all. I therefore talked to practicing engineers in and around Columbus to get some of their opinions on the use of four-way stops. I discovered some interesting similarities and differences with the people in the different communities that I interviewed.

I met with individuals in the city or township that have some type of control over traffic sign and signal installation. Some people are engineers, others are not. The individuals that I interviewed are listed in Figures 10 and 11. The standard interview questions I asked are in Figure 12.

3.1.1 Typical City Organization

In order to understand the process of sign installation, it is important to know something about the general structure within the city government. When an

individual requests the installation of a stop sign, the City Engineer's office will conduct a study to see if the sign is justified according to the Ohio Manual of Uniform Traffic Control Devices (OMUTCD), see Figure 13. For the larger communities, such as the City of Columbus, this is done by the Traffic Engineering office. These engineers are a part of the Service Department within the city, and they are under the Service Director. The Safety Director is at the same organizational level as the Service director and is usually involved in the stop sign installation process. When the initial justification study is complete, the City Engineer or City Traffic Engineer will make a recommendation to the Service Director to install or not to install the requested sign. The Service Director then presents his or her recommendation to the Council who ultimately approves the sign installation.

I found this to be the general set up for all the cities in and around Columbus that I examined. It is important to be familiar with the relationships between the positions in order to understand some of the problems faced by the individuals I interviewed.

3.1.2 The Interview Process

To begin each interview, I asked each individual whether or not he or she has any bias regarding the use of four-way stops as a method of traffic control. This question brought out many thoughts and opinions and made the

interviews very easy to conduct.

All ten people in the Columbus area feel that they are not biased about the use of four-way stops. Some are not particularly fond of them as an engineering solution to a traffic problem, but all said they use them when they meet the warrants. Several individuals also emphasized the importance of avoiding the indiscriminate use of four-way stops, since this fosters a general disrespect for all stop signs.

3.1.3 Four-Way Stops as a Speed Control Device

The problem is four-way stops are not always used properly and in accordance with the OMUTCD warrants. A frequent complaint is of their use as a speed control device.

Mr. Mayeres says that the City of Columbus as a policy does not install any kind of stop sign to control speed, although they get many citizen requests to do so. (Mayeres, 1988)

Mr. Pierce says that Westerville had erroneously installed some four-way stops for speed control. (Pierce, 1988) The existing through-way policy enables the City to designate speed limits on a roadway. One criteria states that roads one mile or more of continuous length should have 35 mph speed limits. This resulted in the installation of stop signs for the sole purpose of breaking up the length of the road so the speed limit could be reduced to 25 mph.

Mr. Jackson says that stop signs are installed in Newark instead of getting the police to enforce the speed limit. (Jackson, 1988)

Mr. Ridgeway has seen four-way stops used near schools as pedestrian protection; they stop traffic at relatively low-volume intersections that do not meet the warrants so children have a better chance of getting across the street without any problems. He does not particularly recommed this course of action, but he does not see any real problem with it either. "You can say a lot of things about the possibility that you are endangering kids. You can give all kinds of arguments, but they do provide pretty good control and I have not really seen any problem with that or any problems with the kids; but you really have to look at individual cases." (Ridgeway, 1988) One of the arguments that Mr. Ridgeway is talking about is the false sense of security it creates for the pedestrians. There is no guarantee that an approaching car will stop at the intersection as a pedestrian is about to enter it.

3.1.4 Political Stop Signs

Four-way stops and stop signs in general should not be installed as a speed control device. This is a well-known traffic engineering principle, yet they are installed for just this reason. It became evident to me fairly early in the interview process that this is largely due to a misconception by the average community resident. The

typical citizen, or the ones requesting signs, seem to believe that installing a stop sign will reduce the speed on their neighborhood streets. The vehicles do have to come to a halt, but it is not a sound engineering justification for installing a sign. Several engineers I spoke to felt that people tend to drive faster between the stops to make up for lost time. Residents in all of the commumities I examined frequently request the installation of stop signs to slow down traffic on their streets. Unfortunately, it is very difficult for the engineers to convince these people that stop signs should not be installed for speed control. When the individual requests for stop signs are recommended for disapproval, the individual making the request will usually go to his or her political representative to seek recourse. This may result in political pressure on the engineers and ultimately the installation of four-way stops for speed control.

Some cities have problems dealing with this political pressure, others do not. The City of Columbus has not installed an unwarranted stop sign as a result of citizen or political pressure. (Mayeres, 1988) Mr. Mayeres says that the Service Director is very supportive of Traffic's recommendations. They also take time to explain to the individual why the sign was disapproved and they try to take some action to solve the original problem that caused the individual to request the sign. The fact that Columbus is a

very large community in comparison to the other communities that I studied may also be a factor.

The City of Worthington created the Traffic Safety Committee to evaluate all types of citizen requests that could affect traffic movement. (Zimomra, 1988) The committee is comprised of the City Engineer, the Director of Public Service, and the Chief of Police. They evaluate the requests based on the OMUTCD, the general feasibility, and the cost. The committee works well together and benefits from the three different areas of expertise. Once the Traffic Safety Committee evaluates the request, they make a recommendation to the City Manager who then writes a traffic order to implement the request if positive action is recommended. The residents are generally satisfied with the results; only two cases in the past five years were brought up to the City Council for further dispute.

The City of Westerville has what I believe is the best arrangement for handling citizen requests for traffic control devices. The City has an local ordinance that requires compliance with the OMUTCD. (Pierce, 1988) When the City Engineer receives a request, he conducts a study to determine if the request is justifiable. When the study is complete, it is simply a matter of comparing pure facts and numbers to see if the warrants are met. It is very difficult for the requestor to dispute the results and it dramatically reduces the manhours invested into each

request.

Unfortuantely, The City of Newark is at the other end of the spectrum. The City Council has final authority for sign installation; the decision-making process is highly susceptible to political pressure. (Roberts, 1988) When a request is made, the engineers conduct a study to see if the warrants are met; usually they are not. The City Engineer recommends disapproval to the Safety Director. The Safety Director usually upholds the recommendation and passes it along to the Council. In the mean time, the citizen making the request goes to their Councilman and complains that their children are not safe. They want a stop sign to slow traffic down. The Council member is usually more interested in the real consequence of being re-elected than in the possibility of a lawsuit if someone gets into an accident at that unwarranted stop, so they overrule the Safety Director's recommendation and have the sign installed. Mr. Jackson says, "Most of the four-way stops that are up in the City of Newark are political stop signs. [It is] political pressure: the people who live in the neighborhood complain to the Councilmen, then the Council will put up a stop sign. It has nothing to do with traffic warrants or anything else. (Jackson, 1988)

The Newark City Engineer and Safety Director have tried to explain to the Council why they should comply with the warrants and the possible legal ramifications if they do

not. But since Newark has never had a lawsuit involving an unwarranted stop sign, it is understandable from the political viewpoint why the Councilmen concern for re-election overrides their concern for obeying the OMUTCD warrants.

Newark can benefit from Westerville's example. A local ordinance requiring compliance with the OMUTCD would take pressure off the Council members and allow the City Engineer and Safety Director to run their operations much more efficiently and effectively. This would completely remove re-election from the issue of stop sign installation.

3.1.5 Other Issues

In addition to the speed control issue, the individuals I interviewed talked about some other issues regarding the use of four-way stops. Some other benefits and drawbacks are the following: first, Mr. Ridgeway believes that four-way stops are a good solution to sight distance problems if there are no other feasible alternatives to improve sight distance at the intersection. (Ridgeway, 1988) Second, Mr. Watterson says that four-way stops have a significantly lower maintenance cost over traffic signals and are therefore a preferable option if a signal is not necessary. (Watterson, 1988) He also mentioned that they are obviously better in the case of a power outage--they continue to function while traffic signals do not. Mr. Mayeres brought up an interesting fact against the use of four-way stops.

He said that in the case of an accident, it is very difficult for the officer at the scene to determine who is at fault, particularly when there are conflicting stories. (Mayeres, 1988) Another factor against the use of four-way stops is that overuse of four-way stops, especially unwarranted stops, creates an general disrespect for all stop signs. Finally, Mr. Roberts does not believe that a four-way stop is effective when there is more than one lane per approach at the intersection. (Roberts, 1988) Newark has two four-lane roads that intersect in front of the fire station. The intersection is controlled by signal, but when the fire department responds to an emergency the lights revert to flashers (effectively a four-way stop). Mr. Roberts says he has witnessed a great deal of confusion at such times. The drivers do not know how to proceed properly through such an intersection.

3.1.6 Driver Confusion at Four-Way Stops

Driver confusion at four-way stops is another area that I discussed with the people I interviewed. I asked each individual if they believe that four-way stops are confusing to the general public. All but one feel that they are not confusing. As long as the intersections are properly signed and the four-way stop type of control is not overused, the drivers seem to understand what is expected of them. Mr. Pierce disagrees. He believes that drivers are never sure who is supposed to proceed through the intersection next,

and there is a lot of indecision at this type of intersection. (Pierce, 1988) When I told him he was the first person I talked to who feels this way, he said he disagrees not only from his experience but from what he has heard the public say and from what he has seen in the accident statistics he has collected on the four-way stops in his community.

The possibility of driver confusion at four-way stops is the only issue with real disagreement. Everyone I interviewed in and around Columbus has the same basic beliefs regarding the use of four-way stops. Mr. Mayeres' closing remarks are a good summary of these basic beliefs:

> "The installation of four-way stop signs is a tool that is designed to handle a specific problem, and that is essentially angle collisions. Like most tools, it has two cutting edges. If you misuse it, it can cause you problems in terms of maintenance, liability, disregard for warranted stop signs, and so it should be used carefully. From a traffic engineering standpoint, I think they are useful; they have a place and they should be used." (Mayeres, 1988)

3.1.7 <u>Inconsistent Use of Four-Way Stops</u>

Since everyone I interviewed believe that four-way stops should only be installed if they are warranted, I am still left with the question of why some communities use them frequently while others do not use them at all, with the exception of Newark's extreme case of political pressure. Perhaps the warrants are not clear. The OMUTCD only addresses the issues of accident data and volume counts. If this criteria is not met then the manual says

the stop is not warranted. What about the case of limited sight distance with no other feasible alternatives? It seems to me that some communities must go by the warrants exactly while others allow for some engineering judgement for cases like the sight distance one. Another factor could be city size. Worthington has the smallest population and what appears to be the highest number of four-way stops. Perhaps it is easier for citizens in the smaller community to contact the decision-makers in their local government and make a request. It may also be easier for the decision-makers to conduct the studies since there is less land area and they are probably more familiar with their streets, even though they do not have the large staffs of the bigger communities.

I questioned Ms. Zimomra, Worthington's Service Director, about the extensive use of four-way stops in her community. She said that most of their four-way stops are at residential intersections and are usually installed as a result of accident statistics. (Zimomra, 1988) Worthington has a relatively high level of commuter traffic. Many drivers cut through the neighborhoods to try to avoid the signals on the primary roads. This type of driver is probably paying less attention to their driving than someone who is in their own neighborhood, so it is understandable that accident data would support the warranting of four-way stop installations. According to Ms. Zimomra, all four-way

stops that are installed in Worthington do meet at least one of the OMUTCD warrants, and it is usually because of accidents rather than traffic volume.

3.1.8 Four-Way Stops in Northern New Jersey

I was raised in a relatively small suburban community like Worthington, but I do not remember driving through any four-way stops in Ramsey, New Jersey. One difference between the communities is Worthington is located in metropolitan Columbus while Ramsey is located in northeastern New Jersey, part of the New York metropolitan area. I thought it would be interesting to talk to some practicing traffic engineers at home and see if I could find out why four-way stops are used so rarely in that area.

I interviewed three individuals, two are employed by Bergen County and one is employed by a consulting firm. All three engineers feel the same way about four-way stops: they do not like them. Bergen County as a policy, does not consider the use of four-way stops as an option, and Mr. Lyon does not recommend their use. Mr. Lyon believes they are more of a detriment due to driver confusion. (Lyon, 1988) He says that drivers do not know that the vehicle to the right has the right-of-way; this indecision contributes unnecessarily to congestion and delay. Mr. Boulding does not think that drivers are confused by four-way stops, but he knows the drivers ignore them. (Boulding, 1988)

All three individuals agree that four-way stops should be eliminated. They all said that if there is not enough volume to justify a signal, then one street should be declared the major street. This can be done by examining total traffic volumes, truck volumes, available sight distance, and any other influencing physical factors.

It is interesting to note that the use of four-way stops was illegal in the State of New Jersey until about ten years ago when the State adopted the Federal MUTCD as the State Manual governing traffic regulations (Figure 14). (Boulding, 1988) Previously, all traffic regulations were individual state laws, specifically administrative laws. When a new law was proposed, the New Jersey Commissioner of Transportation would advertise the proposal, hold a hearing, and then officially adopt it. When the Commissioner adopted the Federal MUTCD, he eliminated the previous traffic laws, one of which said that one street in an intersection had to be designated as a through street. Since the Federal MUTCD allows the use of four-way stops, they were introduced to New Jersey drivers in various communities.

The Commissioner did not, however, relinquish all control over traffic regulations in the State. Traffic regulations on all public street systems--state, county, or municipal--must be approved by the State. This obviously removes the political pressure from the local engineers, but it also significantly increases the time it takes for

installation. For example, a traffic signal can take from one year to eight years from the inception of the idea until the signal installation. This is evidently not an ideal solution either.

3.1.9 Comparison of Opinions

There is a difference in thought between the people I interviewed in the Columbus area and those in New Jersey. The Columbus area representatives are in favor of four-way stops as long as they meet the warrants and are not overused, while the New Jersey engineers do not use them at all. I thought this difference may be due to the regional differences, but then I spoke with Cleveland's Chief Traffic Engineer. He held the same opinions as the New Jersey engineers. Perhaps the difference is due to city size or suburban area. Or perhaps it is just a matter of what types of intersection control people are accustomed to driving. The Columbus area has several four-way stops while both Cleveland and New Jersey use them infrequently. If people are used to driving through four-way stops, it is more likely for them to find this type of control acceptable.

One issue that everyone agreed upon is four-way stops should not be used unless they meet the warrants specified in the MUTCD. When four-way stops are installed for unwarranted and essentially illegal reasons, lawsuits against the city are a real possibility should accidents occur.

3.2 Legal Cases Involving Four-Way Stops

Four-way stops, like any type of traffic control device, must be installed according to the warrants listed in the MUTCD in order to be legal. If the warrants are not met and an accident occurs at that intersection, then the city can be held responsible in a lawsuit. Although this is not a common occurrence, it does happen. The following cases are examples of lawsuits that have been brought against some Ohio communities.

An individual in Dayton received a moving violation and he brought the City to court. (Pierce, 1988) The City was fined $3.9 million because a tree obstructed ten percent of the stop sign at the intersection in question. The City failed to maintain their stop sign and provide the visibility that is suggested in the OMUTCD.

There was a case in Worthington recently where an individual received a moving violation at a four-way stop. (Zimomra) He claimed that the intersection did not meet the OMUTCD warrants and should not have been a four-way stop. He was unable to prove to the court that it did not meet the warrants, so the City of Worthington won the case.

There have been several cases in the City of Cleveland and its surrounding suburbs in the past few years. The City of Cleveland was brought to court for a stop sign that was installed as a speed control device. (Ritz) There was no other justification for the sign's installation, so the

court ordered the City to remove the sign. There was no monetary penalty in this case.

The City of Brook Park, a suburb of Cleveland, was taken to court over 144 illegally installed stop signs posted at 72 intersections. (Hagan, 1981) The individual who filed suit did so after he got no response to his complaints that the signs were not justified. Brook Park had installed the signs as speed control devices. When the case went to court, the judge ordered the city to study the intersections and remove the signs that did not meet the warrants. All but a few signs were removed.

The City of Seven Hills, another Cleveland suburb, was also taken to court because of illegal stop signs. (Jordan, 1983) A resident contended that signs placed in his neighborhood had been placed there illegally. He testified that they were installed to protect school children but the area was not a school zone and the signs did not meet the warrants. The court supported the resident's claim that the signs created more hazards than they solved, and ordered the city to remove the signs. The city also had to pay the plaintiff's legal fees. An interesting note in this case is that the plaintiff recognized that the city was not to blame. He blamed the citizens who pressured the city into installing the signs in violation of the Ohio MUTCD.

3.3 Interviewees and Legal Cases

All of the individuals I interviewed have heard "horror stories" about lawsuits brought against cities and the heavy fines that are incurred as a result of unwarranted traffic control devices. However, most of them have not had a legal case in their community. This lack of experience does not mean there is a lack of concern. Everyone is concerned about the potential lawsuit, especially the individuals who have been pressured to install unwarranted stops. In some cases, a lawsuit would cause the politicians pressuring for unwarranted sign installation to see their error, but that is not what anyone really wants. Lawsuits result from moving violations or accidents. It should not take personal injury or a monetary penalty to convince people to install only warranted stops. It is the traffic engineer's job to do his or her best to provide safe and efficient travel for the motoring public.

GEORGE L. BU[illegible]ZER
Assistant Administrator, City of Columbus, Ohio
4.5 years in current position
22 years engineering experience

SERGEI VLADIMIR JACKSON, JR.
City Engineer, City of Newark, Ohio
14 years engineering experience

CHARLES H. MAYERES
Traffic Operations Engineer, City of Columbus, Ohio
4 years in current position
19 years traffic engineering experience

LEE A. PIERCE
City Engineer, City of Westerville, Ohio
2 years in current position
16 years engineering experience (previously worked in Franklin County Engineers Office, Traffic Division)

JOSEPH A. RIDGEWAY
City Engineer, City of Columbus, Ohio
3 years in current position
27 years engineering experience (24 years in City of Columbus Traffic Division

JAMES G. ROBERTS
Assistant City Engineer, Newark, Ohio
2 years in current position
4 years engineering experience

WILLIAM W. WATTERSON
City Engineer, City of Worthington, Ohio
2.5 years in current position
13 years engineering experience

JOHN A. WOLFE
Service Director. City of Whitehall, Ohio
6 months in current position
30 years experience in City (policeman, Safety Officer)
no engineering training or experience

NANCY J. WRIGHT
Safety Director, Newark, Ohio
4 years in current position (mayoral appointment)
no engineering training or experience

JUDIE ZIMOMRA
Director of Public Service, City of Worthington, Ohio
4.5 years in current position
background in public finance and public administration

Figure 9: Individuals Interviewed, Columbus Area

Northern New Jersey:

ERNEST A. BOULDING
Bergen County Traffic Engineer
30 years in current position
36 years experience in traffic engineering

CRAIG P. GUIMES
Principal Engineer/Traffic, Bergen County Department of Public Works
22 years in current position
22 years experience in traffic engineering

WALTER W. LYON, JR
Chief Engineer for TAMS Consultants (Bloomfield Office) responsible for all civil engineering projects
1 year in current position
25 years engineering experience (20 in traffic)

City of Cleveland, Ohio

DAVID B. RITZ
Chief Traffic Engineer, City of Cleveland

Figure 10: Individuals Interviewed in Northern New Jersey and Cleveland, Ohio

1. Do you have any bias regarding the use of four-way stops?

2. In what instances do feel they are valid?

3. In what situations do you feel they should be avoided?

4. When and where do you use four-way stops in your jurisdiction?

5. Do you think four-way stops are confusing to the general public?
   if yes...any ideas on how to make them less confusing?

6. Have you been pressured (politically, citizen groups) to install four-way stops where you did not think they were justified?

7. Do you know of any legal cases involving four-way stops?

8. Have you ever been called as an expert witness in a legal suit involving four-way stops?

9. Do you have any other information or comments regarding four-way stops?

10. Do you know anyone else I should interview?

Figure 11: Interview Questions

MULTIWAY STOP INSTALLATIONS

The multiway stop installation, in which all approaches to an intersection are stopped, is useful as a safety measure at some locations. It should ordinarily be used only where the volume of traffic on the intersection roads is approximately equal. It should be used sparingly because of the significant increases in delays and operating costs which result from requiring all of the vehicles using the intersection to stop. Unnecessary stops, when the intersection is clear of conflicting movements, lead to general disrespect for stop signs. A traffic control signal is more satisfactory for an intersection with a volume of traffic large enough to meet the appropriate warrants.

Local authorities shall not use the multiway stop installations (in urban areas) unless one of the following warrants is met:

(a) Where traffic signals are warranted and urgently needed, the multiway stop is an interim measure that can be installed quickly to control traffic while arrangements are being made for the signal installation.

(b) **An accident problem,** as indicated by five or more reported accidents of a type susceptible of correction by a multiway stop installation in a 12-month period. Such accidents include right- and left-turn collisions as well as right-angle collisions. Even though the accident warrants are met, a multiway stop installation should not be used until other less restrictive measures are employed. This may consist of parking restrictions, increase in sign size, improvement of sight distance and better advance signing.

(c) Where it is necessary to change the stop pattern at an intersection, the multiway stop may be used as a temporary measure during a transition period.

(d) **Minimum traffic volume:**

1. The total vehicular volume entering the intersection from all approaches must average at least 500 vehicles per hour for any 8 hours of an average day, and

2. The combined vehicular and pedestrian volume from the minor street or highway must average at least 200 units per hour for the same 8 hours, with an average delay to minor street vehicular traffic of at least 30 seconds per vehicle during the maximum hours, but:

3. When the 85-percentile approach speed of the major street traffic exceeds 40 miles per hour, the minimum vehicular volume warrant is 70 percent of the above requirements.

The multiway stop installation should not be used as a permanent treatment on rural highways.

Figure 12: Ohio Manual on Uniform Traffic Control Devices Warrants for Multiway Stops (OMUTCD, 1972)

MULTIWAY STOP SIGNS

The "Multiway Stop" installation is useful as a safety measure at some locations.. It should ordinarily be used only where the volume of traffic on the intersecting roads is approximately equal. A traffic control signal is more satisfactory for an intersection with a heavy volume of traffic.

Any of the following conditions may warrant a multiway STOP sign installation.

1. Where traffic signals are warranted and urgently needed, the multiway stop is an interim measure that can be installed quickly to control traffic while arrangements are being made for the signal installation.

2. An accident problem, as indicated by five or more reported accidents of a type susceptible of correction by a multiway stop installation in a 12-month period. Such accidents include right- and left- turn collisions as well as right-angle collisions.

3. Minimum traffic volumes:

(a) The total vehicular volume entering the intersection from all approaches must average at least 500 vehicles per hour for any 8 hours of an average day, and

(b) The combined vehicular and pedestrian volume from the minor street or highway must average at least 200 units per hour for the same 8 hours, with an average delay to minor street vehicular traffic of at least 30 seconds per vehicle during the maximum hour, but

(c) When the 85-percentile approach speed of the major street traffic exceeds 40 miles per hour, the minimum vehicular volume warrant is 70 percent of the above requirements.

Figure 13: Manual on Uniform Traffic Control Devices (FHWA) Warrants for Installation of Multiway Stops (MUTCD, 1978)

GENERAL DISCUSSION

The four-way stop as a traffic control device has been and continues to be a subject of much controversy. Despite the controversy, there has been limited research in this area.

The research on four-way stops is centered around delay. The earlier studies were concerned with establishing warrants. Following studies focused on capacity, then with the onset of the energy crisis, traffic engineers became concerned with fuel and emissions in addition to delay costs. Now the focus is back to capacity and level of service. The most recent research involves the development of an analytical model, Richardson's M/G/1 queuing model, to predict average delay at four-way stops.

The M/G/1 queuing model uses traffic volume and numbers of lanes to predict average time in the system on an approach at a four-way stop. The model uses values from a 1963 study to predict intermediate values of the average and variance of the service time. The model is validated through comparison to predictions made by a simulation model.

The field work involved in this study examines two basic concepts in the M/G/1 queuing model. First, it checks the accuracy of the headways measured in the 1963 study to see if they are still valid for today's drivers. Since a difference exists, the effect of these values on the model predictions is evaluated. Second, it compares the model predictions of average time in the system to actual values measured in the field.

The field work shows that the M/G/1 queuing model gives reliable predictions of average delay in the system. The revised model that uses the current service time values is more accurate than the model based on the 1963 values. The model can be improved by developing a way to incorporate turn movements into the predictions.

The measured service times provided sufficient information to generate new values for basic four-way stop intersection capacity. The new capacity values are substantially different from the values in the 1985 Highway Capacity Manual. However, a national study should be conducted before these values are adopted, since the field measurements were taken in only one basic geographic area.

It is important to continue research on four-way stops. Practicing traffic engineers use the existing guidance, but there is much variation in the use of four-way stops in the field. Much of the overuse of four-way stops is due to political pressure. An effort should be made to eliminate

the conditions that allow pressure groups to succeed in their efforts to have unwarranted signs installed. This would save time and money, particularly with cases that end up in court.

Four-way stops are a confusing issue. The controversy continues and traffic engineers may never agree on all the aspects of the use of four-way stops. The solution seems to be that there are both good and bad factors associated with four-way stops. The practicing traffic engineer must be aware of them and make the best possible engineering decision. It is therefore important to develop good and accurate guidance.

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APPENDIX

DATA RELEVANT TO CHAPTER II

Table 15: Average Statistical Values for Departure Headway, Intersection 1: Park Road and Worthington-Galena Road

AVERAGE MEAN OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	5.4608	5.0774	4.0468	4.9920
TYPE N	2.7053	2.9467	2.1048	2.6447
TYPE I	3.1929	2.8431	2.6692	2.8608

AVERAGE VARIANCE OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	3.9636	3.8598	3.1709	3.9898
TYPE N	1.6209	0.9669	0.8210	1.2612
TYPE I	1.8859	1.7662	1.0287	1.5722

AVERAGE COUNT (SAMPLE SIZE) OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	77	153	52	282
TYPE N	19	23	15	57
TYPE I	34	64	53	151

Table 16: Average Statistical Values for Departure Headway, Intersection 2: West New England Avenue and Evening Street

AVERAGE MEAN OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	4.8995	4.8208	4.6830	4.8544
TYPE N	2.5758	2.5563	2.6394	2.5811
TYPE I	2.8738	2.8244	2.4100	2.7091

AVERAGE VARIANCE OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	4.3323	4.5336	2.4949	3.9561
TYPE N	1.0468	0.9609	1.0783	1.0072
TYPE I	0.9543	0.9821	0.5522	0.8864

AVERAGE COUNT (SAMPLE SIZE) OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	36	85	43	164
TYPE N	42	139	63	224
TYPE I	27	73	43	143

Table 17: Average Statistical Values for Departure Headway, Intersection 3: West New England Avenue and Oxford Street

AVERAGE MEAN OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	4.9088	4.4855	4.0369	4.5222
TYPE N	2.5980	2.5485	2.7224	2.5837
TYPE I	2.7510	2.7317	2.7708	2.7437

AVERAGE VARIANCE OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	2.4873	2.2244	1.5314	2.2575
TYPE N	0.8888	0.8789	0.7972	0.8718
TYPE I	1.6246	1.0515	1.5080	1.2526

AVERAGE COUNT (SAMPLE SIZE) OF DEPARTURE HEADWAY

	LEFT	THROUGH	RIGHT	TOTAL
TYPE L	41	88	26	155
TYPE N	46	180	41	267
TYPE I	30	104	38	172

Table 18: Average Arrival Rate and Average Time in the System for Each Approach of Each Intersection

AVERAGE ARRIVAL RATE

INTERSECTION	MEAN (sec)	VARIANCE (sec)	COUNT (n=)
PARK & W-G			
North	8.5954	73.6975	211
South	13.6818	209.0723	176
East	23.2840	429.5511	86
West	9.9859	113.7067	301
NE & EVENING			
North	23.4545	638.8700	198
South	24.0593	798.8779	186
East	38.2481	1540.5024	142
West	27.3794	762.5260	231
NE & OXFORD			
North	23.5147	631.1883	254
South	159.6281	38122.2359	37
East	25.7433	674.6884	132
West	25.8058	908.7192	218

AVERAGE TIME IN THE SYSTEM

INTERSECTION	MEAN (sec)	VARIANCE (sec)	COUNT (n=)
PARK & W-G			
North	1.7124	7.4108	102
South	3.3067	16.8966	123
East	1.4880	5.0359	80
West	6.8780	52.1283	215
NE & EVENING			
North	1.6268	4.4244	170
South	1.3297	4.6129	152
East	0.9177	2.6832	134
West	1.4131	5.5898	192
NE & OXFORD			
North	1.2481	4.1712	222
South	0.9252	1.5152	36
East	1.1655	8.5892	102
West	1.3773	9.6947	176